Adaptive Harmony Search Algorithm for Design Code Optimization of Steel Structures

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Abstract. In this chapter an improved version of harmony search algorithm called an adaptive harmony search algorithm is presented. The harmony memory considering rate and pitch adjusting rate are conceived as the two main parameters of the technique for generating new solution vectors. In the standard implementation of the technique, appropriate constant values are assigned to these parameters following a sensitivity analysis for each problem considered. The success of the optimization process is directly related to a chosen parameter value set. The adaptive harmony search algorithm proposed here incorporates a novel approach for adjusting these parameters automatically during the search for the most efficient optimization process. The efficiency of the proposed algorithm is numerically investigated using number of steel frameworks that are designed for minimum weight according to the provisions of various international steel design code specifications. The solutions obtained are compared with those of the standard algorithm as well as those of the other metaheuristic search techniques. It is shown that the proposed algorithm improves performance of the technique and it renders unnecessary the initial selection of the harmony search parameters.

1 Introduction

Design optimization of steel structures is important for structural engineers in today's world due to the fact that while the human population is increasing exponentially, the world resources are diminishing rapidly. More shelters are required to be built for living and more buildings are necessary to be constructed for production. Hence it is of the most importance that structures be designed and constructed by using minimum amount of material available. Optimum structural design algorithms provide a useful tool to steel designers to achieve this goal. These algorithms can be used to design a steel structure such that the design constraints specified by steel design codes are satisfied under the applied loads and the weight or the cost of the steel frame under consideration is the minimum. Formulation of the design optimization of steel structures produces a programming problem where the design variables are discrete in nature. The reason for this is that the steel sections to be adopted for frame members in practice are available

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from a discrete list. Designer has the option of assigning any one of these available steel sections from the list to any one of member groups in the frame either arbitrarily or using his or her previous experience. Once an assignment is carried out for all the member groups in the frame, designer has a candidate solution in her or his hand for the design problem. It then becomes necessary to analyze the frame with these selected steel profiles to find out whether the response of the frame under the external loading is within the limitations set by design codes. If the result of analysis reveals that these limitations are satisfied then the designer has a feasible solution to the frame design problem. It is quite natural that the designer wonders whether there are other solutions would require less steel. As a result of this curiosity the search continues until the designer locates a feasible design which is better then the previously obtained designs in terms of the material required for its construction. It is apparent that this procedure is quite time consuming because quite large number of combinations is possible for the member groups of a frame depending upon the total number of practically available steel sections. For example for a frame where the members are collected in nine groups and that the total number of available steel profile sections is 120, there are 5.16×10^{18} possible combinations each of which can be a possible candidate for the frame under consideration and required to be tried. Some of these combinations may be eliminated by making use of designer's practical experience but still checking the remaining possibilities needs enormous computation time and effort to locate the optimum combination of steel sections. It is apparent that practicing structural designer will have neither time nor resources to carry out this search which covers all the possibilities. He or she will take the decision about steel sections to be used for member groups after few trials. Hence one of the feasible s olutions will be used for the design but not the optimum one.

Obtaining the solution of combinatorial optimization problems described above is not an easy task. Until recently the numbers of solution techniques available in the literature that can be used to determine the optimum solution of discrete programming problems were limited and their efficiency in large size design problems was challenging [1,2]. The emergence of meta-heuristic optimization techniques has opened a new era in obtaining the solution of such programming problems [3-7]. These techniques make use of ideas taken from the nature such as survival of the fittest, immune system or cooling of molten metals through annealing to develop a numerical optimization algorithm. These methods are non-traditional stochastic search and optimization methods and they are very suitable and effective in finding the solution of combinatorial optimization problems. They do not require the gradient information of the objective function and constraints and they use probabilistic transition rules not deterministic rules. They are shown to be quite effective in finding the optimum solution of optimization problems where the design variables are discrete. Among those available in the literature are simulated annealing, evolution strategies, particle swarm optimizer, tabu search method, genetic algorithm and ant colony optimization. As can be understood from their names each technique simulates one particular phenomenon that exists in the nature. There are large numbers of structural optimization procedures available in the literature each is based one of these techniques.

Harmony search algorithm is a new addition to this category of numerical optimization procedures and it simulates a jazz musician's improvisation [8-10]. It resembles an analogy between the attempt to find the harmony in music and the effort to find the optimum solution of an optimization problem. As the aim of a musician is to attain a piece of music with perfect harmony, the task of an optimizer is to come up with the optimum solution that satisfies all the constraints in the problem and minimizes the objective function. Naturally certain rules and parameters are used to transfer this innovative thinking into a numerical optimization technique. For example when a musician is improvising there are three possibilities. A tune can be played from musician's memory or above mentioned tune can be pitch adjusted or a tune can be played totally randomly. Harmony search method is based on these options. It may randomly select a steel section within previously identified and collected group of feasible sections, it may or may not apply pitch adjustment to this section depending on some random rule, or a steel section may randomly be selected from the entire steel sections list. The collected group of feasible solutions is stored in harmony memory matrix. Harmony search method is applied to various structural design optimization problems and found to be quite effective in obtaining their solution.

In this chapter code based design optimization of steel frames is first presented. The mathematical modeling of the discrete optimum design problem of steel frames formulated according to the provisions of Allowable Stress Design code of American Institute of Steel Construction (ASD-AISC) [11], Load and Resistance Factor Design (LRFD-AISC) [12] and British Steel design Code (BS 5950) [13] are described. This is followed by the presentation of the adaptive harmony search method which is an improved version of harmony search algorithm. In this technique two main parameters of the standard harmony search technique that is the harmony memory considering rate and pitch adjusting rate are adjusted automatically during the search procedure. In the standard implementation of the technique appropriate constant values are assigned to these parameters following a sensitivity analysis for each problem considered. The success of the optimization process is directly related to a chosen parameter value set. The adaptive harmony search algorithm presented in this chapter adjusts these parameters automatically during the search for the most efficient optimization process. The efficiency of the adaptive harmony search technique is numerically investigated by considering three design optimization problems of steel frames. The first one is three dimensional 209-member industrial steel frame. The second one is the three dimensional 568-member moment resisting steel frame. The third one is the 1890-member three dimensional braced steel frame. All these frames are designed for minimum weight according to provisions of Allowable Stress Design Code of American Institute of Steel Construction (ASD-AISC). The solutions obtained are compared with those of the standard harmony search algorithm as well as of the other metaheuristic search techniques. It is apparent that the design examples are selected among the real size steel frames that can be found in practice. In the following section the optimum design of the 115-member braced plane frame is carried out according to various international design code specifications namely, ASD-AISC, LRFD-AISC and BS 5950 using adaptive harmony search technique. The results obtained are compared to demonstrate the relationship between the design code used and the optimum solution obtained.

2 Code Based Design Optimization of Steel Frames

The formulation of the design optimization problem of a steel frame according to a steel design code yields itself to a discrete programming problem, if steel profiles for its members are to be selected from available steel sections list. The mathematical model of the design optimization problems depending on three international steel design codes considered in the formulation is described in the following.

2.1 Discrete Optimum Design of Steel Frames to ADS-AISC

Consider a steel structure consisting of *nm* members that are collected in *ng* design groups (variables). If the provisions of ASD-AISC [11] code are to be used in the formulation of the design optimization problem and the design groups are selected from given steel sections profile list, the following discrete programming problem is obtained.

Find a vector of integer values **I** (Eqn. 1) representing the sequence numbers of steel sections assigned to *ng* member groups

$$\mathbf{I}^T = \left[I_1, I_2, ..., I_{ng}\right] \tag{1}$$

to minimize the weight (W) of the frame

$$W = \sum_{i=1}^{ng} m_i \sum_{j=1}^{nt} L_j \tag{2}$$

where m_i is the unit weight of the steel section adopted for member group i, respectively, nt is the total number of members in group i, and L_j is the length of the member j which belongs to group i.

The members subjected to a combination of axial compression and flexural stress must be sized to meet the following stress constraints:

$$if\ \frac{f_a}{F_a} > 0.15; \quad \left[\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}}\right] - 1.0 \le 0 \tag{3}$$

$$\left[\frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}\right] - 1.0 \le 0 \tag{4}$$

$$if\ \frac{f_a}{F_a} \le 0.15;\quad \left[\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}\right] - 1.0 \le 0 \tag{5}$$

If the flexural member is under tension, then the following formula is used instead:

$$\left[\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}\right] - 1.0 \le 0 \tag{6}$$

In Eqns. (3-6), F_y is the material yield stress, and $f_a = (P/A)$ represents the computed axial stress, where A is the cross-sectional area of the member. The computed flexural stresses due to bending of the member about its major (x) and minor (y) principal axes are denoted by f_{bx} and f_{by}, respectively. F'_{ex} and F'_{ey} denote the Euler stresses about principal axes of the member that are divided by a factory of safety of 23/12. F_a stands for the allowable axial stress under axial compression force alone, and is calculated depending on elastic or inelastic buckling failure mode of the member using Formulas 1.5-1 and 1.5-2 given in ASD-AISC [11]. For an axially loaded bracing member whose slenderness ratio exceeds 120, F_a is increased by a factor of $(1.6 - L/200r)$ considering relative unimportance of the member, where L and r are the length and radii of gyration of the member, respectively. The allowable bending compressive stresses about major and minor axes are designated by F_{bx} and F_{by}, which are computed using the Formulas 1.5-6a or 1.5-6b and 1.5-7 given in ASD-AISC [11]. C_{mx} and C_{my} are the reduction factors, introduced to counterbalance overestimation of the effect of secondary moments by the amplification factors $(1 - f_a/F'_e)$. For unbraced frame members, they are taken as 0.85. For braced frame members without transverse loading between their ends, they are calculated from $C_m = 0.6 - 0.4(M_1/M_2)$, where M_1/M_2 is the ratio of smaller end moment to the larger end moment. Finally, for braced frame members having transverse loading between their ends, they are determined from the formula $C_m = 1 + \psi(f_a/F'_e)$ based on a rational approximate analysis outlined in ASD-AISC [11] Commentary-H1, where ψ is a parameter that considers maximum deflection and maximum moment in the member.

For computation of allowable compression and Euler stresses, the effective length factors K are required. For beam and bracing members, K is taken equal to unity. For column members, alignment charts are furnished in ASD-AISC [11] for calculation of K values for both braced and unbraced cases. In this study, however, the following approximate effective length formulas are used based on Dumonteil [14], which are accurate within about -1.0 and +2.0 % of exact results [15]:

For unbraced members:

$$K = \sqrt{\frac{1.6G_A G_B + 4(G_A + G_B) + 7.5}{G_A + G_B + 7.5}} \tag{7}$$

For braced members:

$$K = \frac{3G_A G_B + 1.4(G_A + G_B) + 0.64}{3G_A G_B + 2.0(G_A + G_B) + 1.28} \tag{8}$$

where G_A and G_B refer to stiffness ratio or relative stiffness of a column at its two ends.

It is also required that computed shear stresses (f_v) in members are smaller than allowable shear stresses (F_v), as formulated in Eqn. (9).

$$f_v \leq F_v = 0.40 C_v F_y \tag{9}$$

In Eqn. (9), C_v is referred to as web shear coefficient. It is taken equal to $C_v = 1.0$ for rolled I-shaped members with $h / t_w \leq 2.24 E / F_y$, where h is the clear distance between flanges, E is the elasticity modulus and t_w is the thickness of web. For all other symmetric shapes, C_v is calculated from Formulas G2-3, G2-4 and G2-5 in ANSI/AISC 360-05 [16].

Apart from stress constraints, slenderness limitations are also imposed on all members such that maximum slenderness ratio ($\lambda = KL / r$) is limited to 300 for members under tension, and to 200 for members under compression loads. The displacement constraints are imposed such that the maximum lateral displacements are restricted to be less than *H*/400, and upper limit of story drift is set to be *h*/400, where *H* is the total height of the frame building and *h* is the height of a story.

Finally, we consider geometric constraints between beams and columns framing into each other at a common joint for practicality of an optimum solution generated. For the two beams B1 and B2 and the column shown in Figure 1, one can write the following geometric constraints:

$$\frac{b_{fb}}{b_{fc}} - 1.0 \leq 0 \tag{10}$$

$$\frac{b'_{fb}}{(d_c - 2t_f)} - 1.0 \leq 0 \tag{11}$$

where b_{fb}, b'_{fb} and b_{fc} are the flange width of the beam B1, the beam B2 and the column, respectively, d_c is the depth of the column, and t_f is the flange width of the column. Equation (10) simply ensures that the flange width of the beam B1 remains smaller than that of the column. On the other hand, Eqn. (11) enables that

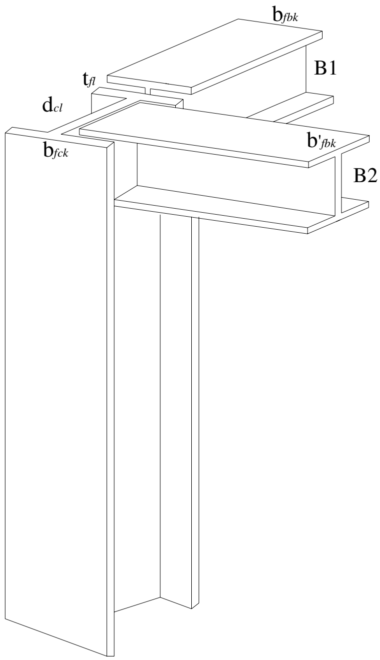


Fig. 1 Beam-column geometric constraints.

flange width of the beam B2 remains smaller than clear distance between the flanges of the column $(d_c - 2t_f)$.

2.2 Discrete Optimum Design of Steel Frames to LRFD-AISC

In the case where the optimum design problem of a steel frame is formulated according to the provisions of LRFD-AISC [12] the following discrete programming problem is obtained.

Find a vector of integer values $\mathbf{I}$ (Eqn. 12) representing the sequence numbers of steel sections assigned to *ng* member groups

$$\mathbf{I}^T = \left[I_1, I_2, ..., I_{ng} \right] \tag{12}$$

to minimize the weight (*W*) of the frame

$$W = \sum_{i=1}^{ng} m_i \sum_{j=1}^{nt} L_j \tag{13}$$

where m_i is the unit weight of the steel section adopted for member group *i*, respectively, *nt* is the total number of members in group *i*, and L_j is the length of the member *j* which belongs to group *i*. The following constraints are required to be imposed according to LRFD-AISC provisions.

$$(\delta_j - \delta_{j-1})/h_j \le \delta_{ju} \qquad j = 1,...., ns \qquad (14)$$

$$\delta_i \le \delta_{iu} \qquad i = 1,...., nd \qquad (15)$$

$$\frac{P_{uk}}{\phi P_{nk}} + \frac{8}{9}\left(\frac{M_{uxk}}{\phi_b M_{nxk}} + \frac{M_{uyk}}{\phi_b M_{nyk}}\right) \le 1 \quad for \frac{P_{uk}}{\phi P_{nk}} \ge 0.2 \qquad k = 1,...., nc \qquad (16)$$

$$\frac{P_{uk}}{2\phi P_{nk}} + \left(\frac{M_{uxk}}{\phi_b M_{nxk}} + \frac{M_{uyk}}{\phi_b M_{nyk}}\right) \le 1 \quad for \frac{P_{uk}}{\phi P_{nk}} < 0.2 \qquad k = 1,...., nc \qquad (17)$$

$$M_{uxt} \le \phi\, M_{nxt} \qquad t = 1,......, nb \qquad (18)$$

$$b_{fb} \le b_{fc} \qquad \text{at all joints} \qquad (19)$$

$$b_{jb} \le (d_c - 2t_f) \qquad \text{at all joints} \qquad (20)$$

where Eqn. (14) represents the inter-story drift of the multi-story frame. δ_j and δ_{j-1} are lateral deflections of two adjacent story levels and h_j is the story height. *ns* is the total number of storys in the frame. Equation (15) defines the displacement restrictions that may be required to include other than drift constraints such as deflections in beams. *nd* is the total number of restricted displacements in the frame. δ_{ju} is the allowable lateral displacement. The allowable lateral displacements are restricted to be less than *H*/400, and upper limit of story drift is set to be *h*/400, where *H* is the total height of the frame building and *h* is the height of a story.

Eqns. (16) and (17) represent strength constraints for doubly and singly symmetric steel members subjected to axial force and bending. If the axial force in member *k* is tensile force, the terms in these equations are given as: P_{uk} is the required axial tensile strength, P_{nk} is the nominal tensile strength, ϕ becomes ϕ_t in the case of tension and called strength reduction factor which is given as 0.90 for yielding in the gross section and 0.75 for fracture in the net section, ϕ_b is the strength reduction factor for flexure given as 0.90, M_{uxk} and M_{uyk} are the required flexural strength, M_{nxk} and M_{nyk} are the nominal flexural strength about major and minor axis of member *k* respectively. It should be pointed out that required flexural bending moment should include second-order effects. LRFD suggests an approximate procedure for computation of such effects which is explained in C1 of LRFD. In the case the axial force in member *k* is compressive force, the terms in Eqns. (16) and (17) are defined as: P_{uk} is the required compressive strength, P_{nk} is the nominal compressive strength, and ϕ becomes ϕ_c which is the resistance factor for compression given as 0.85. The remaining notations in Eqns. (16) and (17) are the same as the definition given above.

The nominal tensile strength of member *k* for yielding in the gross section is computed as $P_{nk} = F_y A_{gk}$ where F_y is the specified yield stress and A_{gk} is the gross area of member *k*. The nominal compressive strength of member *k* is computed as $P_{nk} = A_{gk} F_{cr}$ where $F_{cr} = \left(0.658^{\lambda_c^2}\right) F_y$ for $\lambda_c \le 1.5$ and $F_{cr} = \left(0.877 / \lambda_c^2\right) F_y$ for $\lambda_c > 1.5$ and $\lambda_c = \frac{K\,l}{r\pi}\sqrt{F_y / E}$. In these expressions *E* is the modulus of elasticity, *K* and *l* are the effective length factor and the laterally unbraced length of member *k* respectively.

Equation (18) represents the strength requirements for beams in load and resistance factor design according to LRFD-F2. M_{uxt} and M_{nxt} are the required and the nominal moment about major axis in beam b respectively. ϕ_b is the resistance factor for flexure given as 0.90. M_{nxt} is equal to M_p, plastic moment strength of beam b which is computed as ZF_y where Z is the plastic modulus and F_y is the specified minimum yield stress for laterally supported beams with compact sections. The computation of M_{nxb} for non-compact and partially compact sections is given in Appendix F of LRFD.

Equation (19) is included in the design problem to ensure that the flange width of the beam section at each beam-column connection of story *s* should be less than or equal to the flange width of column section. Equation (20) enables that flange width of the beam B2 remains smaller than clear distance between the flanges of the column $(d_c - 2t_f)$. The notations in Eqns. (19) and (20) are shown in Figure 1.

2.3 Discrete Optimum Design of Steel Frames to BS5950

In case BS5950 [13] is used in formulation of the optimum design problem of a steel frame, the following discrete programming problem is obtained.

Find a vector of integer values $\mathbf{I}$ (Eqn. 21) representing the sequence numbers of steel sections assigned to *ng* member groups

$$\mathbf{I}^T = \left[I_1, I_2, \ldots, I_{ng}\right] \tag{21}$$

to minimize the weight (*W*) of the frame

$$W = \sum_{i=1}^{ng} m_i \sum_{j=1}^{nt} L_j \tag{22}$$

where m_i is the unit weight of the steel section adopted for member group *i*, respectively, *nt* is the total number of members in group *i*, and L_j is the length of the member *j* which belongs to group *i*. The following constraints are required to be imposed according to BS5950 provisions.

$$(\delta_j - \delta_{j-1}) / h_j \le \delta_{ju} \qquad j = 1, \ldots, ns \tag{23}$$

$$\delta_i \leq \delta_{iu} \qquad i = 1,...., nd \qquad (24)$$

$$\frac{F_k}{A_{gk}\, p_y} + \frac{M_{xk}}{M_{cxk}} + \frac{M_{yk}}{M_{cyk}} \leq 1 \qquad k = 1,...., nc \qquad (25)$$

or

$$\frac{F_k}{A_{gk}\, P_{ck}} + \frac{m_k M_{xk}}{M_{bk}} + \frac{m_k M_{yk}}{p_y Z_{yk}} \leq 1 \qquad k = 1,...., nc \qquad (26)$$

$$M_{xn} \leq M_{cxn} \qquad t = 1,......, nb \qquad (27)$$

$$b_{fb} \leq b_{fc} \qquad \text{at all joints} \qquad (28)$$

$$b_{jb} \leq \left(d_c - 2t_f\right) \qquad \text{at all joints} \qquad (29)$$

Equation (23) represents the inter-story drift of the multi-story frame. δ_j and δ_{j-1} are lateral deflections of two adjacent story levels and h_j is the story height. *ns* is the total number of storys in the frame. Equation (24) defines the displacement restrictions that may be required to include other than drift constraints such as deflections in beams. *nd* is the total number of restricted displacements in the frame. δ_{ju} is the allowable lateral displacement. BS 5950 limits the horizontal deflection of columns due to unfactored imposed load and wind loads to height of column/300 in each story of a building with more than one story. δ_{iu} is the upper bound on the deflection of beams which is given as span/360 if they carry plaster or other brittle finish.

Equation (25) defines the local capacity check for beam-columns. F_k , M_{xk} and M_{yk} are the applied axial load and moments about the major and minor axis at the critical region of member k respectively. A_{gk} is the gross cross sectional area, and p_y is the design strength of the steel. M_{cxk} and M_{cyk} are the moment capacities about major and minor axis of member *k*. nc is the total number of beam-columns in the frame.

Equation (26) represents the simplified approach for the overall buckling check for beam-columns. m_k is the equivalent uniform moment factor of member k given in table 18 of BS 5950. M_{bk} is the buckling resistance moment capacity for member k about its major axis computed from clause 4.3.7 of the code. Z_{yk} is the elastic section modulus about the minor axis of member *k*. P_{ck} is the compression strength obtained from the solution of quadratic Perry-Robertson formula given in appendix C.1 of BS 5950. It is apparent that computation of the compressive strength of a compression member requires its effective length. This can be automated by using Jackson and Moreland monograph for frame buckling [17].

The relationship for the effective length of a column in a swaying frame is given as:

$$\frac{(\gamma_1\gamma_2)(\pi/k)^2 - 36}{6(\gamma_1+\gamma_2)} = \frac{\pi/k}{\tan(\pi/k)} \tag{30}$$

where k is the effective length factor and γ_1 and γ_2 are the relative stiffness ratio for the compression member which are given as:

$$\gamma_1 = \frac{\sum I_{c1}/\ell_{c1}}{\sum I_{b1}/\ell_{b1}} \quad \text{and} \quad \gamma_2 = \frac{\sum I_{c2}/\ell_{c2}}{\sum I_{b2}/\ell_{b2}} \tag{31}$$

The subscripts c and b refer to the compressed and restraining members respectively and the subscripts 1 and 2 refer to two ends of the compression member under investigation. The solution of the nonlinear equation (30) for k results in the effective length factor for the member under consideration. The Eqn. (30) has the following form for non-swaying frames.

$$\frac{\gamma_1\gamma_2}{4}\left(\frac{\pi}{k}\right)^2 + \left(\frac{\gamma_1+\gamma_2}{2}\right)\left(1-\frac{\pi/k}{\tan(\pi/k)}\right) + \frac{2\tan(\pi/2k)}{\pi/k} = 1 \tag{32}$$

The notations in the remaining inequalities (28) and (29) are the same as those defined in inequalities (19) and (20).

3 Adaptive Harmony Search Algorithm

Harmony search method is a recent meta-heuristic technique that is shown to be effective and robust in obtaining the optimum solution of discrete programming problems. Its use in structural optimization and computational mechanics is still new. Among the few numbers of studies Lee and Geem [9] applied the method to determine the optimum design of plane and space trusses with continuous design variables. The method is used in the optimum design of steel frames with discrete variables by Değertekin [18] and Saka [19] where the design problem is formulated according to LRFD-AISC and BS5950 respectively. Later the same technique is employed in the optimum design of grillage systems [20, 21]. It is shown by Saka [22, 23] that harmony search algorithm can also be used in shape optimization problems. In this study harmony search method has successfully determined the optimum height of a geodesic dome in addition to pipe section designations for its members. It is demonstrated within these studies that harmony search method was a rapid and effective method for optimum design of structural systems where the number of design variables was relatively small. However, a comprehensive performance evaluation of harmony search method carried out at Hasançebi et al. [24, 25] in real size large scale structural optimization problems has shown that this conclusion were only true for small size problems. In this study the technique is compared with other meta-heuristic algorithms and found out that in large scale design optimization problems the technique has demonstrated slow convergence rate and heavier optimum designs. Hence it became necessary to suggest some improvements in the standard harmony search method so that the above mentioned discrepancy can be eliminated and the method demonstrates similar performance with other meta-heuristic techniques in the case of large scale design problems. With this amendment an improved technique called adaptive harmony search method is formulated and proposed in Hasançebi et al. [26].

In standard harmony search method there are two parameters known as harmony memory considering rate ($hmcr$) and pitch adjusting rate (par) that play an important role in obtaining the optimum solution. These parameters are assigned to constant values that are arbitrarily chosen within their recommended ranges by Geem [27-29] based on the observed efficiency of the technique in different problem fields. It is observed through the application of the standard harmony search method that the selection of these values is problem dependent. While a certain set of values yields a good performance of the technique in one type of design problem, the same set may not present the same performance in another type of design problem. Hence it is not possible to come up with a set of values that can be used in every optimum design problem. In each problem a sensitivity analysis is required to be carried out to determine what set of values results a good performance. Adaptive harmony search method eliminates the necessity of finding the best set of parameter values. It adjusts the values of these parameters automatically during the optimization process. Before initiating the design process, a set of steel sections selected from an available profile list are collected in a design pool. Each steel section is assigned a sequence number that varies between 1 to total number of sections (N_{sec}) in the list. It is important to note that during optimization process selection of sections for design variables is carried out using these numbers. The basic components of the adaptive harmony search algorithm can now be outlined as follows.

3.1 Initialization of a Parameter Set

Harmony search method uses four parameters values of which are required to be selected by the user. This parameter set consists of a harmony memory size (hms), a harmony memory considering rate ($hmcr$), a pitch adjusting rate (par) and a maximum search number ($N_{\max}$). Out of these four parameters, $hmcr$ and par are made dynamic parameters in adaptive harmony search method that vary from one solution vector to another. They are set to initial values of $hmcr^{(0)}$ and $par^{(0)}$ for all the solution vectors in the initial harmony memory matrix. In the standard harmony search algorithm these parameters are treated as static quantities, and they are assigned to suitable values chosen within their recommended ranges of $hmcr \in [0.70, 0.95]$ and $par \in [0.20, 0.50]$ [27-29].

3.2 Initialization of Harmony Memory Matrix

A harmony memory matrix **H** given in Eqn. (33) is randomly generated. The harmony memory matrix simply represents a design population for the solution of a problem under consideration, and incorporates a predefined number of solution vectors referred to as harmony memory size (hms). Each solution vector (harmony vector, $\mathbf{I}^i$) consists of ng design variables, and is represented in a separate row of the matrix; consequently the size of $\mathbf{H}$ is $(hms \times ng)$.

$$\mathbf{H} = \begin{bmatrix} I_1^1 & I_2^1 & \dots & I_{ng}^1 \\ I_1^2 & I_2^2 & \dots & I_{ng}^2 \\ \dots & \dots & \dots & \dots \\ I_1^{hms} & I_2^{hms} & \dots & I_{ng}^{hms} \end{bmatrix} \begin{matrix} \phi(\mathbf{I}^1) \\ \phi(\mathbf{I}^2) \\ \dots \\ \phi(\mathbf{I}^{hms}) \end{matrix} \tag{33}$$

3.3 Evaluation of Harmony Memory Matrix

The structural analysis of each solution is then performed with the set of steel sections selected for design variables, and responses of each candidate solution are obtained under the applied loads. The objective function values of the feasible solutions that satisfy all problem constraints are directly calculated from Eqn. (2). However, infeasible solutions that violate some of the problem constraints are penalized using external penalty function approach, and their objective function values are calculated according to Eqn. (34).

$$\phi = W\left[1 + \alpha\left(\sum_i g_i\right)\right] \tag{34}$$

In Eqn. (34), ϕ is the constrained objective function value, g_i is the *i*-th problem constraint and α is the penalty coefficient used to tune the intensity of penalization as a whole. This parameter is set to an appropriate static value of $\alpha = 1$ in the numerical examples. Finally, the solutions evaluated are sorted in the matrix in the descending order of objective function values, that is, $\phi(\mathbf{I}^1) \leq \phi(\mathbf{I}^2) \leq \ldots \leq \phi(\mathbf{I}^{hms})$.

3.4 Generating a New Harmony Vector

In harmony search algorithm the generation of a new solution (harmony) vector is controlled by two parameters ($hmcr$ and par) of the technique. The harmony memory considering rate ($hmcr$) refers to a probability value that biases the algorithm to select a value for a design variable either from harmony memory or from the entire set of discrete values used for the variable. That is to say, this parameter decides in what extent previously visited favorable solutions should be considered in comparison to exploration of new design regions while generating new solutions. At times when the variable is selected from harmony memory, it is checked whether this value should be substituted with its very lower or upper neighboring one in the discrete set. Here the goal is to encourage a more explorative search by allowing transitions to designs in the vicinity of the current solutions. This phenomenon is known as pitch-adjustment in HS, and is controlled by pitch adjusting rate parameter (par). In the standard algorithm both of these parameters are set to suitable constant values for all harmony vectors generated regardless of whether an exploitative or explorative search is indeed required at a time during the search process. On the contrary, in the adaptive algorithm a new set of values is sampled for *hmcr* and *par* parameters each time prior to improvisation (generation) of a new harmony vector, which in fact forms the basis for the algorithm to gain adaptation to varying features of the design space. Accordingly, to generate a new harmony vector in the algorithm proposed, a two-step procedure is followed consisting of (i) sampling of control parameters, and (ii) improvisation of the design vector.

3.4.1 Sampling of Control Parameters

For each harmony vector to be generated during the search process, first a new set of values are sampled for *hmcr* and *par* control parameters by applying a logistic normal distribution based variation to the average values of these parameters within the harmony memory matrix, as formulated in Eqns. (35 and 36).

$$(hmcr)^k = \left(1 + \frac{1-(hmcr)'}{(hmcr)'} . e^{-\gamma . N(0,1)}\right)^{-1} \tag{35}$$

$$(par)^k = \left(1 + \frac{1-(par)'}{(par)'} . e^{-\gamma . N(0,1)}\right)^{-1} \tag{36}$$

In Eqns. (35) and (36), $(hmcr)^k$ and $(par)^k$ represent the sampled values of the control parameters for a new harmony vector. The notation $N(0,1)$ designates a normally distributed random number having expectation 0 and standard deviation 1. The symbols $(hmcr)'$ and $(par)'$ denote the average values of control parameters within the harmony memory matrix, obtained by averaging the corresponding values of all the solution vectors within the $\mathbf{H}$ matrix, that is,

$$(hmcr)' = \frac{\sum_{i=1}^{\mu}(hmcr)^i}{(hms)} \quad , \quad (par) = \frac{\sum_{i=1}^{\mu}(par)^i}{(hms)} \tag{37}$$

Finally, the factor γ in Eqns. (35) and (36) refers to the learning rate of control parameters, which is recommended to be selected within a range of [0.25, 0.50]. In the numerical examples this parameter is set to 0.35.

In the proposed implementation, for each new vector a probabilistic sampling of control parameters is motivated around average values of these parameters $(hmcr)'$ and $(par)'$ observed in the $\mathbf{H}$ matrix. Considering the fact that the harmony memory matrix at an instant incorporates the best *hms* solutions sampled thus far during the search, the idea here is to encourage forthcoming vectors to be sampled with values that the search process has taken the most advantage in the past. The use of a logistic normal distribution provides an ideal platform in this sense because not only it guarantees the sampled values of control parameters to lie within their possible range of variation, i.e., [0, 1], but also it permits occurrence of small variations around $(hmcr)'$ and $(par)'$ more frequently than large ones. Accordingly, sampled values of control parameters mostly fall within close vicinity of the average values, yet remote values are occasionally promoted to check alternating demands of the search process.

3.4.2 Improvisation of the Design Vector

Upon sampling of a new set of values for control parameters, the new harmony vector $\mathbf{I}^k = \left[I_1^k, I_2^k, .., I_{ng}^k\right]$ is improvised in such a way that each design variable is selected at random from either harmony memory matrix or the entire discrete set. Which one of these two sets is used for a variable is determined probabilistically in conjunction with harmony memory considering rate $(hmcr)^k$ parameter of the solution. To implement the process a uniform random number r_i is generated between 0 and 1 for each variable I_i^k. If r_i is smaller than or equal to $(hmcr)^k$, the variable is chosen from harmony memory in which case it is assigned any value from the i-th column of the $\mathbf{H}$ matrix, representing the value set of the variable in hms solutions of the matrix (Eqn. 38). Otherwise (if $r_i > (hmcr)^k$), an arbitrary value is assigned to the variable from the entire design set.

$$I_i^k = \begin{cases} I_i^k \in \left\{I_i^1, I_i^2, ..., I_i^{hms}\right\} & if\ r_i \le (hmcr)^k \\ I_i^k \in \left\{1, .., N_{sec}\right\} & if\ r_i > (hmcr)^k \end{cases} \tag{38}$$

If a design variable attains its value from harmony memory, it is checked whether this value should be pitch-adjusted or not. In pitch adjustment the value of a design variable ($I_i^{k'}$) is altered to its very upper or lower neighboring value obtained by adding ± 1 to its current value. This process is also operated probabilistically in conjunction with pitch adjusting rate $(par)^k$ parameter of the solution, Eqn. (37). If not activated by $(par)^k$, the value of the variable does not change. Pitch adjustment prevents stagnation and improves the harmony memory for diversity with a greater change of reaching the global optimum.

$$I_i^{k'} = \begin{cases} I_i^k \pm 1 & if\ r_i \le (par)^k \\ I_i^k & if\ r_i > (par)^k \end{cases} \tag{39}$$

3.5 Update of the Harmony Memory and Adaptivity

After generating the new harmony vector, its objective function value is calculated as per Eqn. (34). If this value is better (lower) than that of the worst solution in the harmony memory matrix, it is included in the matrix while the worst one is discarded out of the matrix. It follows that the solutions in the harmony memory matrix represent the best (hms) design points located thus far during the search. The harmony memory matrix is then sorted in ascending order of objective function value. Whenever a new solution is added into the harmony memory matrix, the $(hmcr)'$ and $(par)'$ parameters are recalculated using Eqn. (37). This way the harmony memory matrix is updated with the most recent information required for an efficient search and the forthcoming solution vectors are guided to make their own selection of control parameters mostly around these updated values. It should be underlined that there are no single values of control parameters that lead to the most efficient search of the algorithm throughout the process unless the design domain is completely uniform. On the contrary, the optimum values of control parameters have a tendency to change over time depending on various regions of the design space in which the search is carried out. The update of the control parameters within the harmony memory matrix enables the algorithm to catch up with the varying needs of the search process as well. Hence the most advantageous values of control parameters are adapted in the course of time automatically (i.e., by the algorithm itself), which plays the major role in the success of *adaptive* harmony search method discussed in this chapter.

3.6 Termination

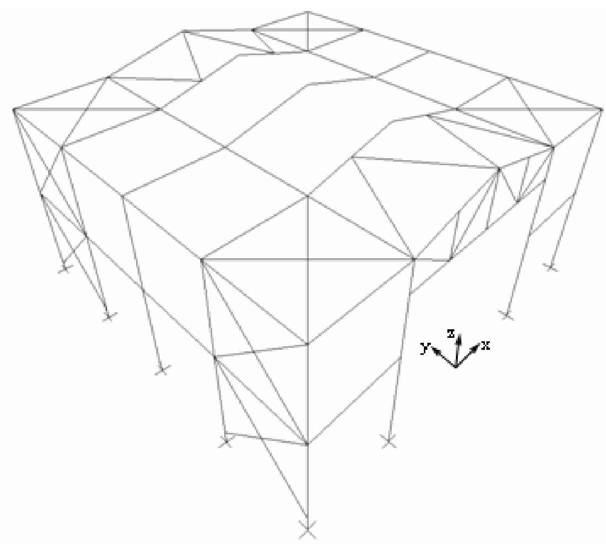
The steps 3.4 and 3.5 are iterated in the same manner for each solution sampled in the process, and the algorithm terminates when a predefined number of solutions ($N_{\max}$) is sampled.

4 Performance Evaluation of Adaptive Harmony Search Method

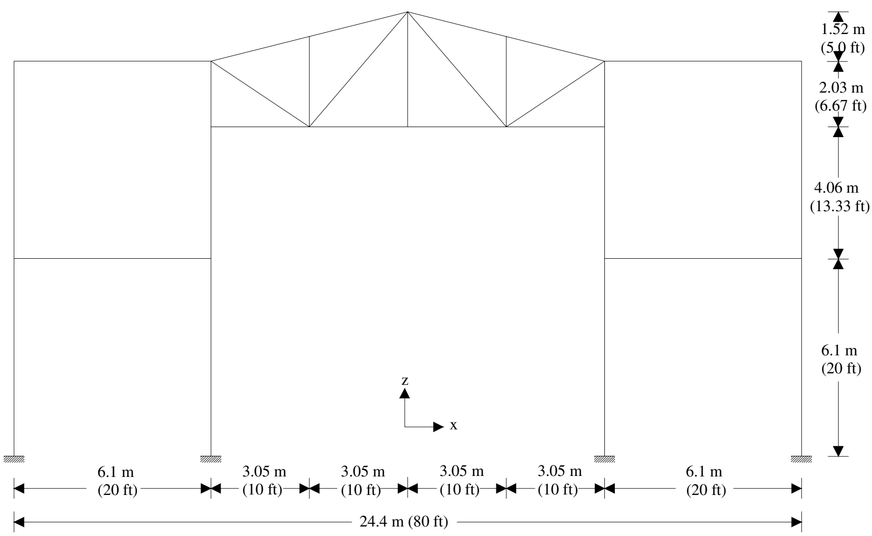
Performance of the adaptive harmony search algorithm presented is evaluated in the optimum design of three real size steel frames. These are 209-member industrial factory building, 568-member unbraced space steel frame and 1860-member braced space steel frame, respectively. The topology and geometry of each frame and the loadings considered in their designs are described in the relevant sections below. The design constraints in these three problems are arranged according to ASD-AISC design code specifications and the following material properties of the steel are used: modulus of elasticity (E) = 203893.6 MPa (29000 ksi) and yield stress (F_y) = 253.1 MPa (36 ksi). Each frame is designed using both standard and adaptive harmony search algorithms and the performance of the techniques is compared.

4.1 209-Member Industrial Factory Building

The first design example is an industrial factory building with 100 joints and 209 members. Shown in Figure 2 are the plan, side and 3D views of this structure. The main system of the structure consists of five identical frameworks lying 6.1 m (20 ft) apart from each other in x-z plane. Each framework consists of two side frames and a gable roof truss in between them as depicted in Figure 2 (b). The lateral stability against wind loads in x-z plane is provided with columns fixed at the base along with the rigid connections of the side frames. Hence, all the beams and columns in the side frames are designed as moment-resisting axial-flexural members.

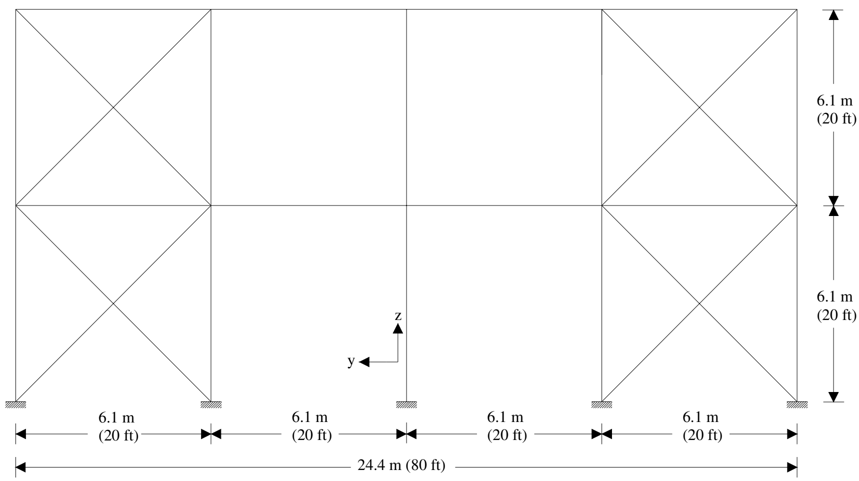


a) 3D view

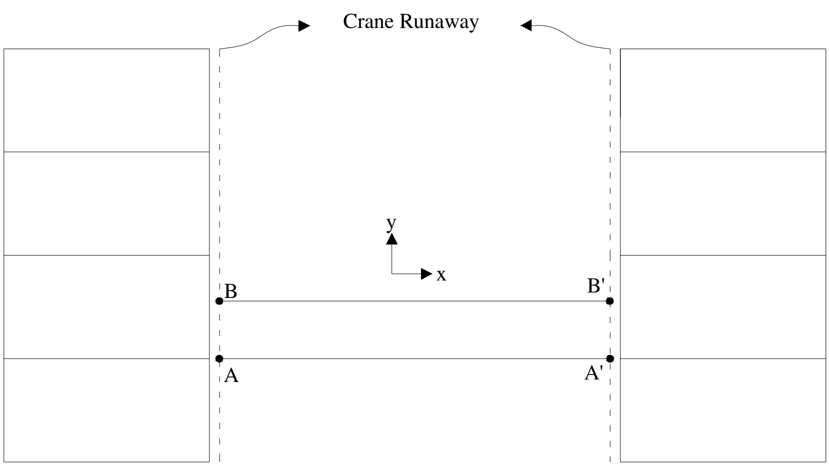


b) Front view

Fig. 2 209-member industrial factory building.

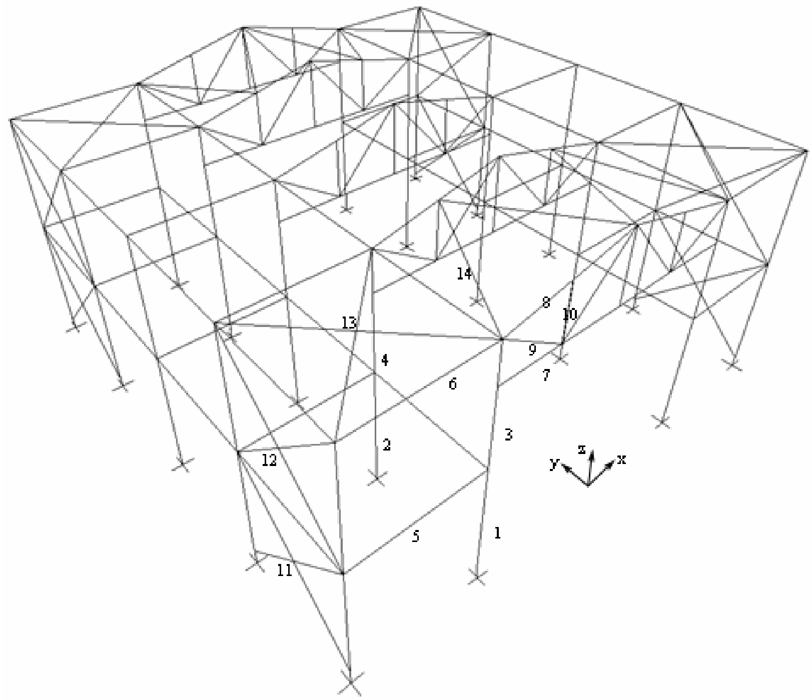


c) Side view



d) First floor plan view

Fig. 2 (*continued*)



e) Member grouping

Fig. 2 (*continued*)

The gable roof truss, on the other hand, is designed to transmit only axial forces through pin-jointed connections, and hence the web and chord members in the gable roof are all designed as axial members. For longitudinal stability (along y-axis) of the structure, bracing is provided in the end bays in the walls and the roof. By employing the symmetry of the structure and fabrication requirements of structural members, the total of 209 members are collected in 14 member groups (independent size variables). The member grouping details are presented in Table 1 and Figure 2 (e).

Three different types of loads are considered for the design of the industrial building; namely dead, crane and wind loads. A design dead load of 1.2 kN/m^2 is assumed to be acting on both floors of the side frames, resulting in uniformly distributed loads of 14.63 kN/m (1004.55 lb/ft) and 7.32 kN/m (502.27 lb/ft) on the interior and exterior beams of the side frames. The dead weights of the gable roofs are neglected due to relatively light weight of these components. The crane load is modeled as two pairs of moving live loads acting on both sides of the crane runway beams as shown in Figure 2 (d). Each pair consists of a concentrated load of 280 kN (62.9 kip) and a couple moment of 75 kN.m (5532 kip.ft). In the study the crane load is represented in two distinct load cases referred to as CL1 and CL2 by choosing two different positions for the crane on its runway. In CL1, the crane is positioned at points A and A′ as shown in Figure 2 (d) to create maximum effect on the second framework. In CL2, however, it is positioned in the middle of the runway beam between the second and third frameworks (shown as B and B′ in Figure 2 (d)) to maximize response in the beams directed along y-axis.

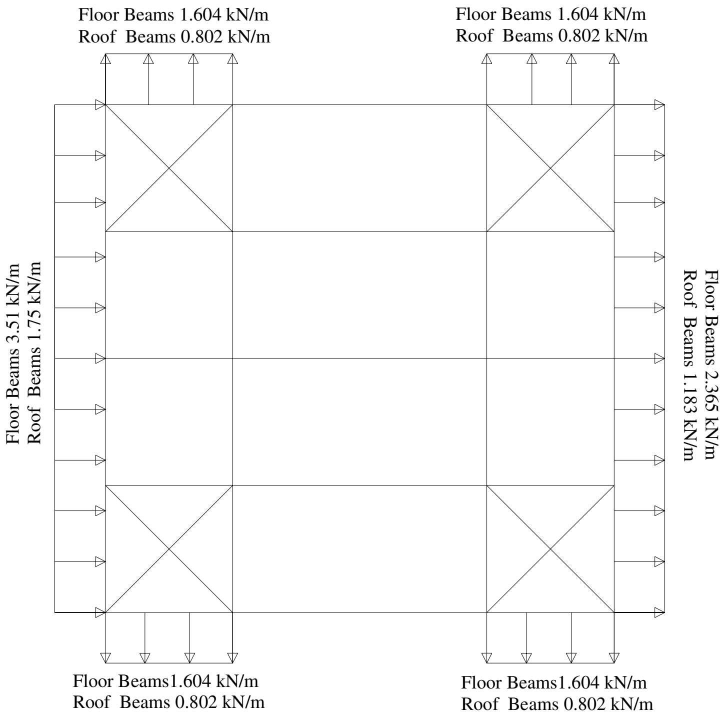
Table 1 Member grouping details for 209-member industrial factory building.

Member	Group Name	Member	Group name
1	1st floor external columns	8	Truss top chord
2	1st floor internal columns	9	Truss web diagonals
3	2nd floor external columns	10	Truss web verticals
4	2nd floor internal columns	11	1st floor wall braces
5	1st floor beams	12	2nd floor wall braces
6	2nd floor beams	13	Floor frames braces
7	Truss bottom chord	14	Floor truss braces

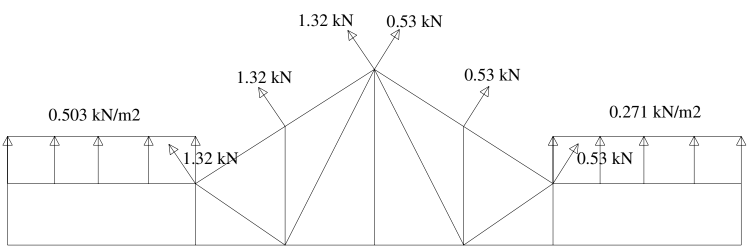
Only the wind in the x-direction is considered for design and the corresponding wind forces are calculated based on a basic wind speed of $V = 46.94$ m/s (105 mph) in line with the prescriptions described in ASCE 7-05 [30], which is discussed in the following example. Two load cases referred to as WL1 and WL2 are generated depending on the sign of the internal wind pressure exerted on the external faces of the building, as shown in Figure 3. In both cases, it is assumed that wind causes a positive compression pressure on windward face, while it causes a negative suction effect on leeward face as well as on side walls of the building. In WL1 the suction effect is considered for the entire roof surface, whereas in WL2 one part of the roof is subjected to compression pressure. From amongst the five load cases (DL, CL1, CL2, WL1 and WL2), a total of six load combinations are generated for the strength design of each structural member in accordance with ASD-AISC [11] specification, as follows:

(i) 1.0DL + 1.0CL1
(ii) 1.0DL + 1.0CL1 + 1.0WL1
(iii) 1.0DL + 1.0CL1 + 1.0WL2
(iv) 1.0DL + 1.0CL2
(v) 1.0DL + 1.0CL2 + 1.0WL1
(vi) 1.0DL + 1.0CL2 + 1.0WL2

All members are sized using the standard sections in AISC. Accordingly, the beam and column members are selected from wide-flange sections (W), and side wall and roof bracings are selected from back to back equal leg double angle sections. The combined stress, stability and geometric constraints are imposed according to the provisions of ASD-AISC [11]. In addition, displacements of all the joints in x and y directions are limited to 3.43 cm (1.25 in), and the upper limit of inter-story drifts is set to 1.52 cm (0.6 in).



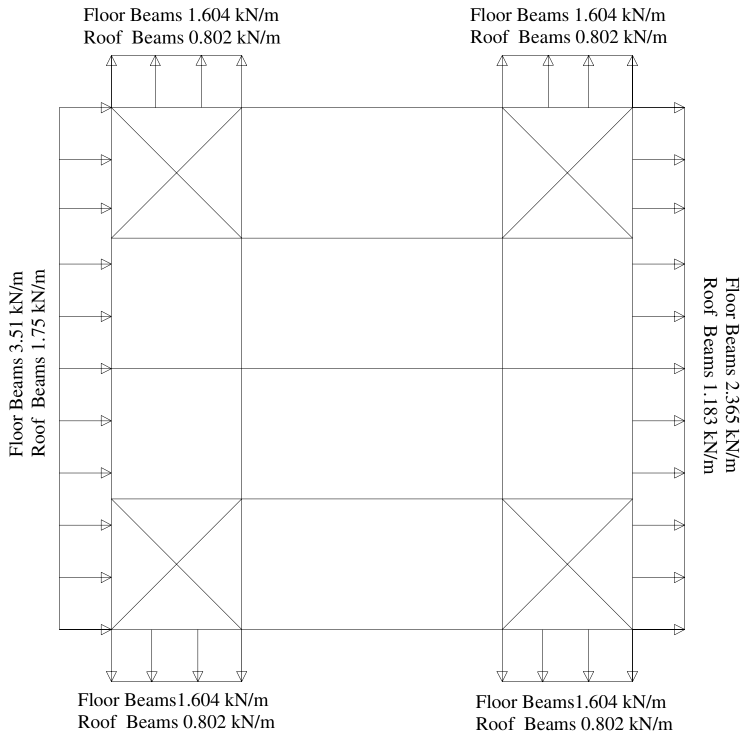
i- wind loads on floor and roof beams (case 1)



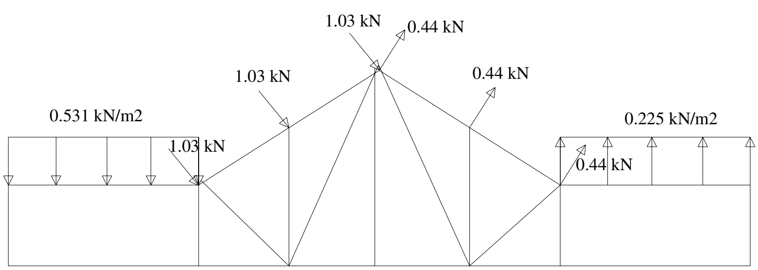
ii- wind loads on the roof (case 1)

a) The first wind load case (WL1)

Fig. 3 The two wind load cases considered for the design of 209-member industrial factory building.



i- wind loads on floor and roof beams (case 2)



ii- wind loads on the roof (case 2)

b) The second wind load case (WL2)

Fig. 3 (*continued*)

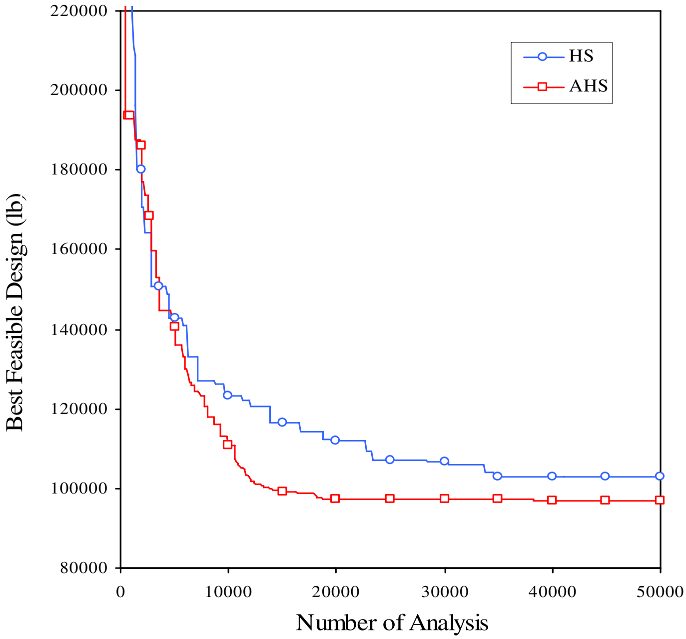


Fig. 4 The design history for 209-member industrial factory building.

The industrial factory building described above is designed by using standard and adaptive harmony search techniques. The harmony memory size *hms*, harmony memory considering rate *hmcr* and pitch adjustment rate *par* is taken as 50, 0.90 and 0.30 in the standard harmony search method while these parameters are adjusted dynamically in the adaptive harmony search technique. The maximum number of iterations is taken as 50000 in both algorithms in order to provide equal opportunity for both algorithms for attaining the global optimum. The design history of both runs is shown in Figure 4. It is apparent from the figure that adaptive harmony method exhibits a better convergence rate and obtains lighter frame. The minimum weight of the steel frame is attained as 46685.83kg by the standard harmony search method while the same weight is obtained as 44053.45kg by the adaptive harmony search algorithm which is 5.6 % lighter. It is also apparent from the figure that adaptive harmony search method approaches to the vicinity of the minimum weight in early iterations of the optimum design process while the standard harmony search method reduces the frame weight in a steady manner until the end of the design process. The steel section designations determined by the both methods for each member groups of the frame are given in Table 2.

Table 2 Optimum designs obtained with standard and adaptive harmony search methods for 209-member industrial factory building.

Group Number	Standard Harmony Search Method		Adaptive Harmony Search Method	
	Ready Section	Area, (cm^2) (in^2)	Ready Section	Area, (cm^2) (in^2)
1	W8X31	58.90 (9.13)	W8X31	58.90 (9.13)
2	W12X40	76.13 (11.8)	W10X39	74.19 (11.5)
3	W8X31	58.90 (9.13)	W12X26	49.35 (7.65)
4	W8X40	75.48 (11.7)	W8X40	75.48 (11.7)
5	W24X62	117.42 (18.2)	W24X62	117.42 (18.2)
6	W12X26	49.35 (7.65)	W10X26	49.09 (7.61)
7	2L2.5X2X3/16	10.44 (1.62)	2L2X2X1/8	6.25 (0.97)
8	2L2X2X1/8	6.25 (0.97)	2L2X2X1/8	6.25 (0.97)
9	2L3X3X3/16	14.06 (2.18)	2L3X3X3/16	14.06 (2.18)
10	2L3X2.5X5/16	20.90 (3.24)	2L2X2X1/8	6.25 (0.97)
11	2L6X6X7/16	65.81 (10.2)	2L6X6X5/16	47.09 (7.30)
12	2L6X6X3/8	56.26 (8.72)	2L6X6X5/16	47.09 (7.30)
13	2L6X6X5/16	47.09 (7.30)	2L6X6X5/16	47.09 (7.30)
14	2L6X6X5/16	47.09 (7.30)	2L5X5X5/16	39.09 (6.06)
Weight	46685.83kg (102924.73 lb)		44053.45kg (97121.3 lb)	

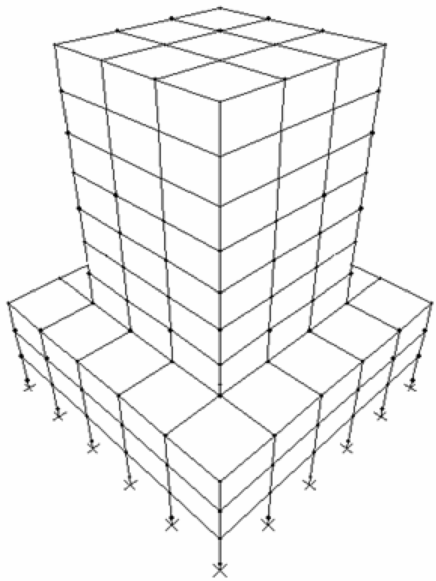
4.2 568-Member Unbraced Space Steel Frame

The second design example shown in Figures 5 (a-d) is a 10-story unbraced space steel frame consisting of 256 joints and 568 members. This problem has been formerly studied in Hasançebi et al. [25] to evaluate the performance of various meta-heuristic search techniques in real size optimum design of steel frameworks. The objective in this problem is then to compare the performance of adaptive harmony search method with those of other meta-heuristic search techniques.

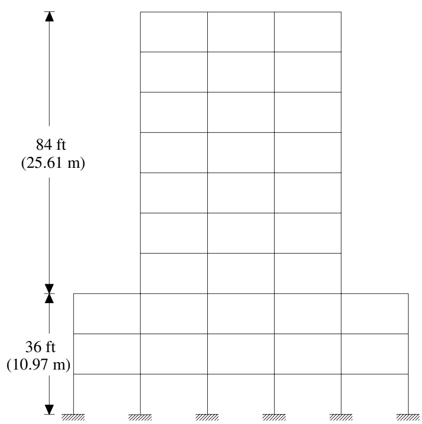
The columns in a story are collected in three member groups as corner columns, inner columns and outer columns, whereas beams are divided into two groups as inner beams and outer beams. The corner columns are grouped together as having the same section in the first three stories and then over two adjacent stories thereafter, as are inner columns, outer columns, inner beams and outer beams. This results in a total of 25 distinct member groups as shown in Figure 5 (d). The columns are selected from the complete W-shape profile list consisting of 297 ready sections, whereas a discrete set of 171 economical sections selected from W-shape profile list based on area and inertia properties is used to size beam members.

The frame is subjected to various gravity loads in addition to lateral wind forces. The gravity loads acting on floor slabs cover dead (DL), live (LL) and snow (SL) loads, which are applied as uniformly distributed loads on the beams using load distribution formulas developed for slabs. All the floors, except the roof, are subjected to a design dead load of 2.88 kN/m^2 (60.13 lb/ft^2) and a design live load of 2.39 kN/m^2 (50 lb/ft^2). The beams of the roof level are subjected to the design dead load plus snow load. The design snow load is computed using the following equation in ASCE 7-05 [30]:

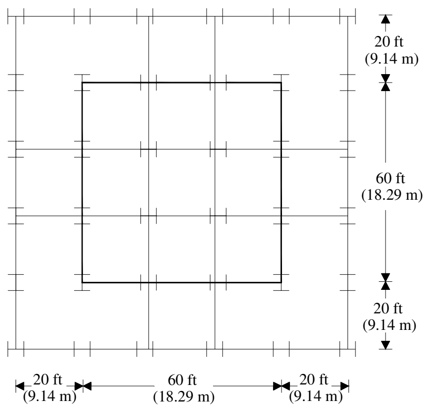
$$p_s = 0.7 C_s C_e C_t I p_g \tag{40}$$



a) 3D view

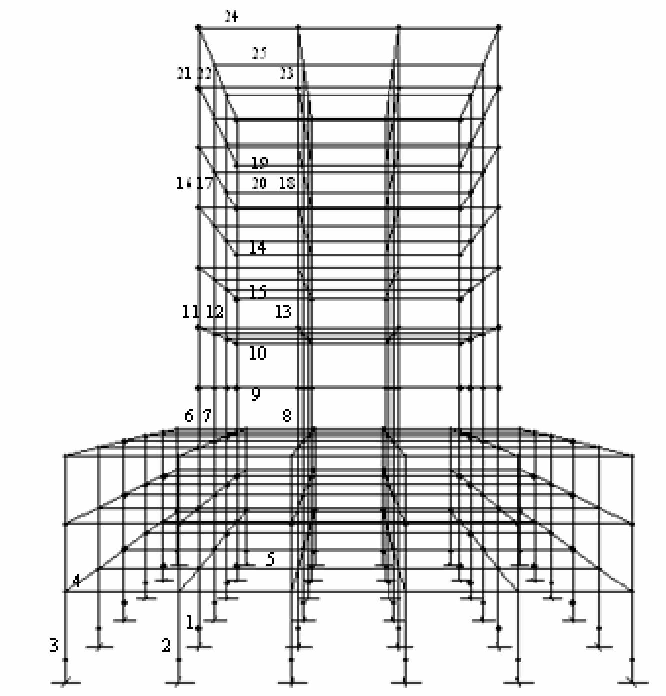


b) Elevation view



c) Plan view

Fig. 5 568-member unbraced space steel frame.



d) Member grouping

* 1st group: inner columns, 2nd group: side columns, 3rd group: corner columns 4th group: outer beams 5th group: inner beams, and so forth.

Fig. 5 (*continued*)

where p_s is the design snow load in kN/m^2, C_s is the roof slope factor, C_e is the exposure factor, C_t is the temperature factor, I is the importance factor, and p_g is the ground snow load. For a heated residential building having a flat and fully exposed roof, these factors are chosen as follows: $C_s = 1.0$, $C_e = 0.9$, $C_t = 1.0$, $I = 1.0$, and $p_g = 1.20$ kN/m^2 (25 lb/ft^2), resulting in a design snow load of 1.20 kN/m^2 (25 lb/ft^2). The resulting gravity loading (GL) on the beams of the roof and floors is tabulated in Table 3.

The wind loads (WL) are applied as uniformly distributed lateral loads on the external beams of the frame located at windward and leeward facades at every floor level. They are also computed according to ASCE 7-05 [30] using the following equation:

$$p_w = (0.613 K_z K_{zt} K_d V^2 I)(GC_p) \tag{41}$$

Table 3 Gravity loading on the beams of 568-member unbraced space steel frame.

Beam Type	Uniformly Distributed Load	
	Outer Beams	Inner Beams
	kN/m (lb/ft)	kN/m (lb/ft)
Roof beams	7.38 (505.879)	14.77 (1011.74)
Floor beams	10.72 (734.20)	21.44 (1468.40)

Table 4 Wind loading on 568-member unbraced space steel frame.

Floor No	Windward	Leeward
	kN/m (lb/ft)	kN/m (lb/ft)
1	1.64 (112.51)	1.86 (127.38)
2	1.88 (128.68)	1.86 (127.38)
3	2.10 (144.68)	1.86 (127.38)
4	2.29 (156.86)	1.86 (127.38)
5	2.44 (167.19)	1.86 (127.38)
6	2.57 (176.13)	1.86 (127.38)
7	2.69 (184.06)	1.86 (127.38)
8	2.79 (191.21)	1.86 (127.38)
9	2.89 (197.76)	1.86 (127.38)
10	1.49 (101.90)	0.93 (63.69)

where p_w is the design wind pressure in kN/m^2, K_z is the velocity exposure coefficient, K_{zt} is the topographic factor, K_d is the wind direction factor, V is the basic wind speed, G is the gust factor, and C_p is the external pressure coefficient. Assuming that the building is located in a flat terrain with a basic wind speed of $V = 46.94$ m/s (105 mph) and exposure category B, the following values are used for these parameters: $K_{zt} = 1.0$, $K_d = 0.85$, $I = 1.0$, $G = 0.85$, and $C_p = 0.8$ for windward face and -0.5 for leeward face. The calculated wind loads at every floor level are presented in Table 4.

The gravity and wind forces are combined under two loading conditions. In the first loading condition, the gravity loading is applied with the wind loading acting along x-axis (1.0GL + 1.0WL-x), whereas in the second one wind loading is acted along y-axis (1.0GL + 1.0WL-y). The combined stress, stability, displacement and geometric constraints are imposed according to the provisions of ASD-AISC [11].

The optimum design of the unbraced space steel frame described above is carried out using the adaptive harmony search algorithm as well as six different meta-heuristic techniques. These meta-heuristic techniques are evolutionary strategies (ES), tabu search optimization (TSO), simulated annealing (SA), ant colony optimization (ACO), simple genetic algorithm (SGA) and particle swarm optimizer (PSO). In each optimization technique the number of iterations is taken as 50000 in order to allow equal opportunity to every technique to grasp the global

Table 5 Comparison of optimum designs obtained by various meta-heuristic search techniques for 568-member unbraced space steel frame.

Member groups	Ready Sections in designs obtained with each metaheuristic optimization technique						
	ESs	AHS	TSO	SA	ACO	SGA	PSO
1	W14X193	W14X176	W14X193	W14X193	W14X193	W14X193	W14X159
2	W8X48	W14X48	W8X48	W8X48	W8X48	W8X48	W24X76
3	W10X39	W10X39	W8X40	W8X40	W10X45	W10X39	W10X39
4	W10X22	W10X22	W10X22	W10X22	W10X22	W10X26	W10X22
5	W21X50	W24X55	W21X50	W21X44	W21X50	W21X50	W24X55
6	W10X54	W12X65	W10X54	W12X65	W14X61	W18X76	W12X72
7	W14X109	W14X145	W14X120	W14X145	W14X120	W14X109	W27X146
8	W14X176	W14X159	W14X159	W14X145	W40X192	W40X192	W27X217
9	W18X40	W14X30	W21X44	W24X68	W18X35	W18X40	W18X40
10	W18X40	W18X40	W18X40	W24X55	W18X40	W21X50	W18X40
11	W10X49	W10X54	W10X45	W10X49	W12X58	W12X65	W18X71
12	W14X90	W14X90	W14X90	W14X90	W12X96	W21X111	W21X101

Table 5 (*continued*)

13	W14X109	W14X120	W12X120	W14X120	W12X136	W12X152	W14X176
14	W14X30	W14X34	W21X44	W16X36	W12X30	W12X30	W14X34
15	W16X36	W18X40	W16X36	W16X40	W21X44	W16X40	W21X44
16	W12X45	W8X31	W10X33	W12X40	W8X58	W14X68	W12X65
17	W12X65	W12X65	W12X65	W12X65	W18X76	W18X76	W10X68
18	W10X22	W18X35	W14X34	W12X26	W12X35	W8X28	W12X35
19	W12X79	W12X79	W12X79	W12X72	W10X88	W10X88	W12X79
20	W14X30	W14X30	W14X30	W16X36	W14X30	W16X36	W14X38
21	W8X35	W10X22	W10X39	W8X24	W8X58	W8X48	W10X39
22	W10X39	W10X45	W12X45	W10X49	W8X40	W14X34	W8X31
23	W8X31	W8X31	W12X35	W8X24	W8X31	W12X30	W12X96
24	W8X18	W10X22	W6X20	W12X26	W8X24	W8X21	W12X26
25	W14X30	W12X26	W12X26	W12X26	W16X45	W18X35	W12X26
Weight, kg (lb)	228588.33 (503953.63)	232301.20 (512139.16)	235167.52 (518458.35)	238756.51 (526370.76)	241470.31 (532353.70)	245564.80 (541380.54)	253260.23 (558346.15)

optimum. The design history of each run by each technique is shown in Figure 6 and the minimum weights as well as W-section designations obtained for each members group is given in Table 5. Inspection of the minimum weights reveals the fact that the lightest frame is attained by the evolutionary strategies and the optimum result obtained by the adaptive harmony search algorithm is the second best among all the meta-heuristic algorithms considered in this study. This clearly indicates that the enhancements carried out in the standard harmony search method have certainly improved the performance of the technique. In fact the optimum design attained by the standard harmony search method for the same frame was 259072.31 kg (571159.66 lb) as given in [25] which was the heaviest among all. The minimum weight found in this study is only 1.6% heavier than the one obtained by evolutionary strategies algorithm.

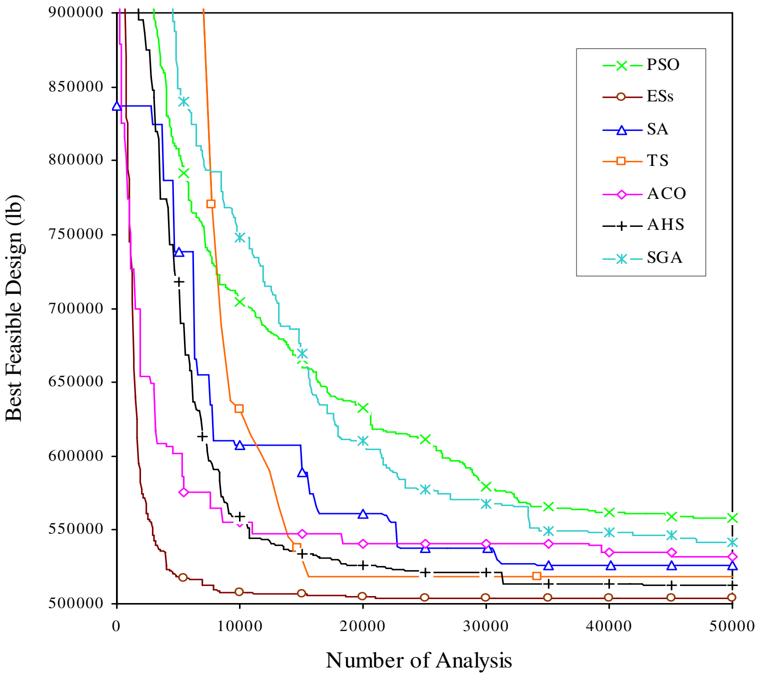
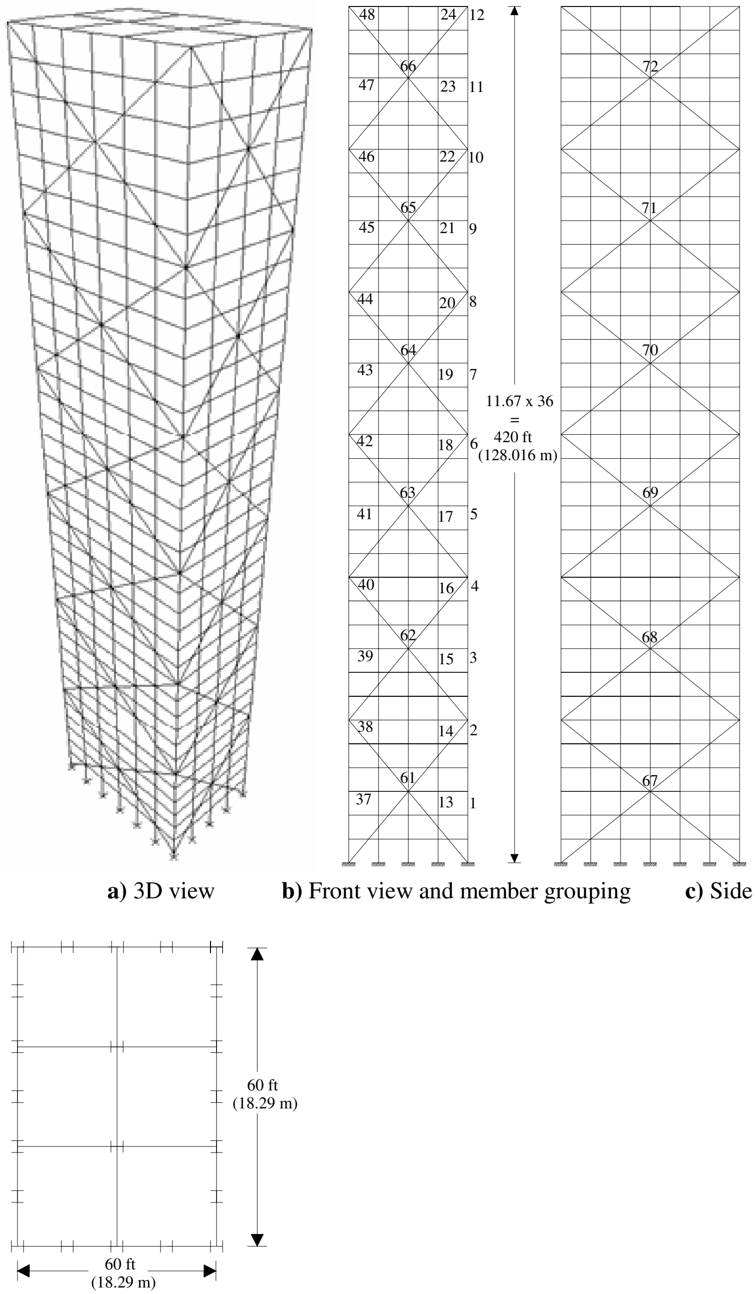


Fig. 6 The design history for meta-heuristic search algorithms used in the optimum design of 568-member unbraced space steel frame.

4.3 1860-Member Braced Space Steel Frame

The last design example considered in this section is 36-story braced space steel frame consisting of 814 joints and 1860 members. The side, plan and 3D views of the frame as well as member grouping details are shown in Figures 7 (a-d). An economical and effective stiffening of the frame against lateral forces is achieved through exterior diagonal bracing members located on the perimeter of the building, which also participate in transmitting the gravity forces.



a) 3D view **b)** Front view and member grouping **c)** Side view

d) Plan view.

Fig. 7 1860-member braced space steel frame.

Table 6 Wind loading on 1860-member braced space steel frame.

Floor	Windward kN/m (lb/ft)	Leeward kN/m (lb/ft)
1	2.05 (140.64)	3.57 (244.70)
2	2.50 (171.44)	3.57 (244.70)
3	2.81 (192.49)	3.57 (244.70)
4	3.05 (208.98)	3.57 (244.70)
5	3.25 (222.74)	3.57 (244.70)
6	3.42 (234.65)	3.57 (244.70)
7	3.58 (245.22)	3.57 (244.70)
8	3.72 (254.75)	3.57 (244.70)
9	3.85 (263.47)	3.57 (244.70)
10	3.96 (271.52)	3.57 (244.70)
11	4.07 (279.02)	3.57 (244.70)
12	4.18 (286.04)	3.57 (244.70)
13	4.27 (292.66)	3.57 (244.70)
14	4.36 (298.92)	3.57 (244.70)
15	4.45 (304.87)	3.57 (244.70)
16	4.53 (310.55)	3.57 (244.70)
17	4.61 (315.97)	3.57 (244.70)
18	4.69 (321.18)	3.57 (244.70)
19	4.76 (326.18)	3.57 (244.70)
20	4.83 (330.99)	3.57 (244.70)
21	4.90 (335.64)	3.57 (244.70)
22	4.97 (340.13)	3.57 (244.70)
23	5.03 (344.48)	3.57 (244.70)
24	5.09 (348.69)	3.57 (244.70)
25	5.15 (352.78)	3.57 (244.70)
26	5.21 (356.76)	3.57 (244.70)
27	5.27 (360.62)	3.57 (244.70)
28	5.32 (364.39)	3.57 (244.70)
29	5.37 (368.06)	3.57 (244.70)
30	5.43 (371.65)	3.57 (244.70)
31	5.48 (375.14)	3.57 (244.70)
32	5.53 (378.56)	3.57 (244.70)
33	5.58 (381.90)	3.57 (244.70)
34	5.62 (385.18)	3.57 (244.70)
35	5.67 (388.38)	3.57 (244.70)
36	2.86 (195.76)	1.79 (122.35)

Table 7 Gravity loading on the beams of 1860-member braced steel space frame.

Beam Type	Uniformly Distributed Load, kN/m (lb/ft)		
	Dead Load	Live Load	Snow Load
Roof beams	22.44 (1536.66)	N.A	5.88 (402.50)
Floor beams	22.44 (1536.66)	18.66 (1277.78)	N.A

The wide-flange (W) profile list consisting of 297 ready sections is used to size column members, while beams and diagonals are selected from discrete sets of 171 and 147 economical sections selected from wide-flange profile list based on area and inertia properties in the former, and on area and radii of gyration properties in the latter. The 1860 frame members are collected in 72 different member groups, considering the symmetry of the structure and practical fabrication requirements. That is, the columns in a story are collected in three member groups as corner columns, inner columns and outer columns, whereas beams are divided into two groups as inner beams and outer beams. The corner columns are grouped together as having the same section over three adjacent stories, as are inner columns, outer columns, inner beams and outer beams. Bracing members on each facade are designed as three-story deep members, and two bracing groups are specified in every six stories.

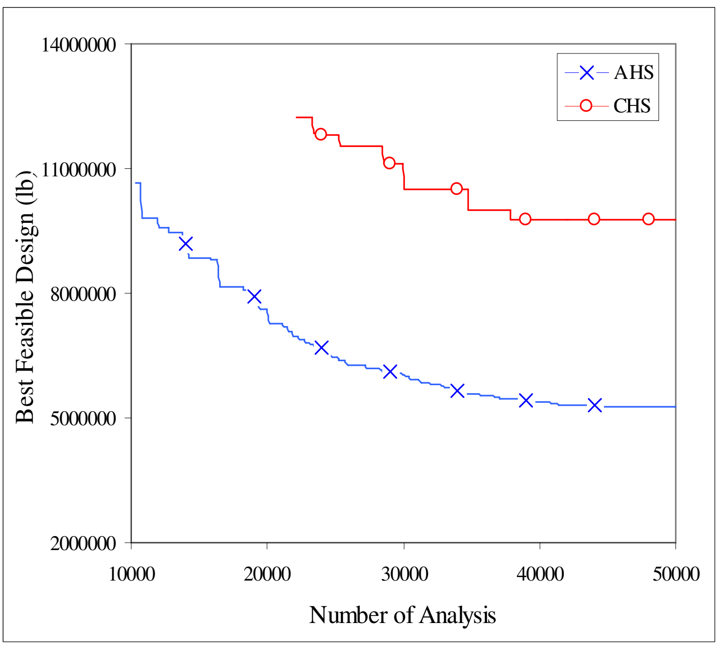


Fig. 8 The design history for standard and adaptive harmony search methods used in the optimum design of 1860-member braced space steel frame.

Table 8 Comparison of optimum designs obtained with classical and adaptive harmony search methods for 1860-member braced steel space frame.

Member Group	Standard Harmony Search Method			Member Group	Adaptive Harmony Search Method		
	Ready Section	Member Group	Ready Section		Ready Section	Member Group	Ready Section
1	W27X258	37	W18X35	1	W24X370	37	W12X14
2	W40X268	38	W30X148	2	W12X336	38	W16X26
3	W24X450	39	W18X35	3	W12X305	39	W14X22
4	W24X250	40	W36X210	4	W12X190	40	W14X26
5	W27X146	41	W10X30	5	W14X193	41	W27X102
6	W36X848	42	W36X160	6	W24X117	42	W16X31
7	W33X201	43	W14X30	7	W12X79	43	W18X35
8	W33X201	44	W33X141	8	W12X79	44	W18X35
9	W14X455	45	W12X16	9	W40X244	45	W40X167
10	W36X359	46	W12X26	10	W18X86	46	W14X22
11	W33X201	47	W33X152	11	W12X96	47	W24X68
12	W24X176	48	W21X50	12	W8X28	48	W16X26
13	W30X581	49	W21X57	13	W33X424	49	W24X62
14	W36X798	50	W33X387	14	W40X436	50	W27X94
15	W14X665	51	W40X167	15	W40X324	51	W24X68
16	W36X798	52	W30X235	16	W36X280	52	W24X76
17	W36X393	53	W30X90	17	W33X318	53	W24X76
18	W36X848	54	W27X84	18	W33X291	54	W30X116

Table 8 (*continued*)

19	W36X848	55	W33X141	19	W40X277	55	W27X94
20	W36X848	56	W36X527	20	W24X250	56	W21X83
21	W33X424	57	W40X167	21	W36X260	57	W30X90
22	W36X848	58	W33X152	22	W33X291	58	W44X198
23	W36X848	59	W44X248	23	W27X235	59	W44X285
24	W40X480	60	W30X124	24	W12X170	60	W24X68
25	W36X848	61	W14X605	25	W14X665	61	W14X455
26	W36X798	62	W14X730	26	W36X798	62	W14X398
27	W36X527	63	W36X328	27	W36X720	63	W40X328
28	W36X848	64	W30X173	28	W33X619	64	W14X233
29	W14X500	65	W14X176	29	W40X531	65	W14X109
30	W27X281	66	W21X166	30	W36X439	66	W12X72
31	W36X798	67	W14X311	31	W27X494	67	W40X328
32	W36X848	68	W33X387	32	W33X619	68	W14X283
33	W36X848	69	W36X300	33	W21X364	69	W14X233
34	W40X324	70	W40X249	34	W40X297	70	W40X192
35	W36X527	71	W40X249	35	W36X245	71	W40X192
36	W36X798	72	W30X261	36	W14X283	72	W14X132
Weight	4438172.37 kg (9784496.01 lb)			2383604.61 kg (5254949.08 lb)			

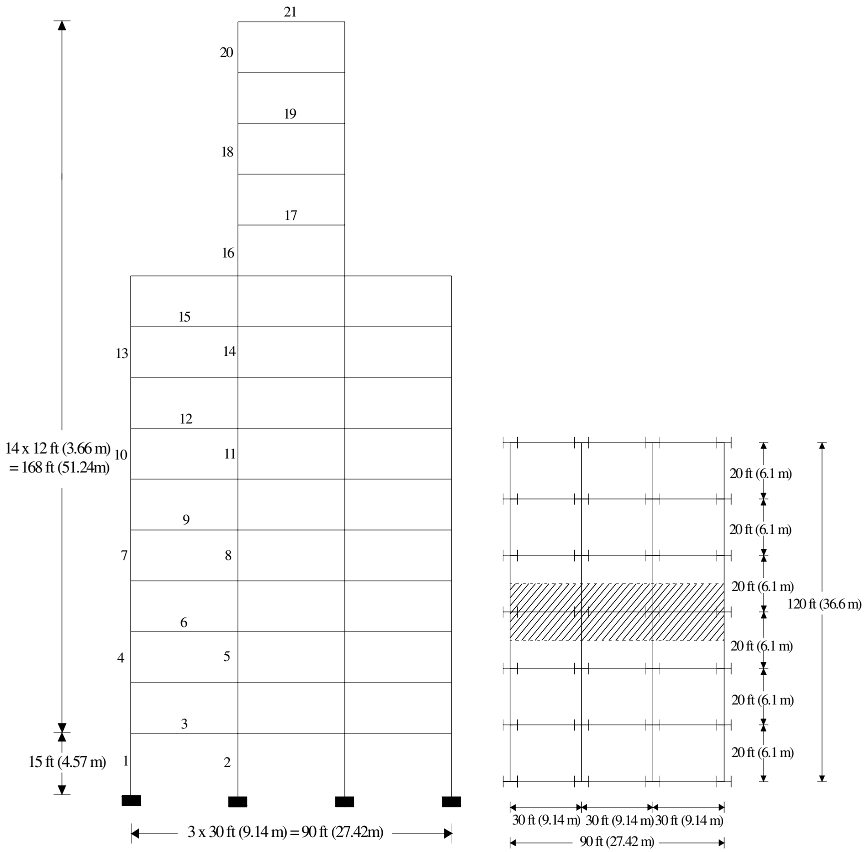
The 1860-member braced space steel frame is subjected to two loading conditions of combined gravity and wind forces. These forces are computed as per ASCE 7-05 based on the following design values: a design dead load of 2.88 kN/m^2 (60.13 lb/ft^2), a design live load of 2.39 kN/m^2 (50 lb/ft^2), a ground snow load of 1.20 kN/m^2 (25 lb/ft^2) and a basic wind speed of 55.21 m/s (123.5mph). Lateral (wind) loads acting at each floor level on windward and leeward faces of the frame are tabulated in Table 6 and the gravity loading on the beams of roof and floors is given in Table 7. In the first loading condition, gravity loads are applied together with wind loads acting along x-axis (1.0 GL + 1.0WL-x), whereas in the second one they are applied with wind loads acting along y-axis (1.0 GL + 1.0WL-y). The combined stress, stability and geometric constraints are imposed according to the provisions of ASD-AISC. The joint displacements in x and y direction are restricted to 32.0 cm (12.6 in) which is obtained as height of frame/400. Furthermore, story drift constraints are applied to each story of the frame which is equal to height of each story/400.

The 1860-member braced space steel frame is designed separately by using both standard and adaptive harmony search method. In the standard harmony search method the harmony memory size, harmony memory considering rate and pitch adjustment rate are taken as 50, 0.90 and 0.10 respectively. The maximum number of iteration is 50000. The design history of both runs is shown in Figure 8 and the optimum designs obtained by the both algorithm is given in Table 8. The minimum weight for the frame is determined as 2383604.61 kg by the adaptive harmony search method while standard harmony search algorithm arrived at 4438172.37 kg which is 46.3% heavier. It is apparent that in optimum design problems where the number of design variables relatively large, standard harmony search method do not perform well and adaptive harmony search technique dissipates this drawback. Figure 8 clearly demonstrates the better performance of the adaptive harmony search method and verifies the above fact.

5 Comparison of Code Based Optimum Designs

Figure 9 shows plan and elevation views of a 85-member moment resisting planar steel frame, which actually represents one of the interior frameworks of a steel building along the short side. The 85 members are grouped into total of 21 independent size variables to satisfy practical fabrication requirements, such that the exterior columns are grouped together as having the same section over two adjacent stories, as are interior columns and beams, as indicated in Figure 9.

The frame is only subjected to gravity loads, which are computed as per ASCE 7-05 [30] based on the following design values: a design dead load of 2.88 kN/m^2 (60.13 lb/ft^2), a design live load of 2.39 kN/m^2 (50 lb/ft^2) and a ground snow load of 1.20 kN/m^2 (25 lb/ft^2). The unfactored distributed gravity loads on the beams of the roof and floors are tabulated in Table 9. The load and combination factors are applied according to each code specification used to size the frame members, as follows: 1.0DL + 1.0L + 1.0SL for ASD-AISC; 1.2DL + 1.6LL + 0.5SL for LRFD-AISC; and 1.4DL + 1.6LL + 1.6SL for BS5950.



a) elevation **b)** plan

Fig. 9 85-member unbraced planar steel frame.

Table 9 Gravity loading on the beams of 85-member unbraced planar steel frame.

Beam Type	Uniformly Distributed Load, kN/m (lb/ft)		
	Dead Load	Live Load	Snow Load
Roof beams	7.47 (512.22)	N.A.	1.96 (134.17)
Floor beams	7.47 (512.22)	6.21 (425.93)	N.A.

In the ASD-AISC and LRFD-AISC code applications the wide-flange (W) profile list consisting of 297 ready sections is used to size column members, while beams are selected from discrete sets of 171 economical sections selected from wide-flange profile list based on area and inertia properties in the optimum design of the frame. In the case of British Code it is common practice to use universal beam (UB) sections for beams and universal column (UC) sections for columns of steel frames. Among the steel sections list 64 universal beam sections starting from 914x419x388UB to 254x102x28UB and 32 universal column sections starting from 356x406x634UC to 152x152x23UC are selected to constitute the discrete set consists of 96 steel sections from which the design algorithm selects the sectional designations for the frame members. The stress, strength and stability requirements of all members are imposed according to provisions of each code specification employed, as outlined in Section 2. In addition, the top story drift and inter-story drifts are limited to a maximum value of $H/400$ and $h/400$, respectively, where H is the total building height and h is the story height. These limitations are somewhat little different in the BS5950 they are given as $H/300$ and $h/300$ where H and h are the same as the previous definitions.

The optimum design of the 85-member frame is carried out according to the each design code provisions using adaptive harmony search algorithm presented in the preceding sections. The optimum designs obtained in each case are given in Table 10. Among three design codes LRFD-AISC attains the lightest frame under the design loading considered in this study. The minimum weight of the frame is determined as 32868.54 kg by LRFD-AISC, 33011 kg by BS5950 and 47472.66 kg by ASD-AISC. The minimum weights found by BS5950 and LRFD-AISC is quite close to each other while the one determined by ASD-AISC is 44.4% heavier than the one attained by LRFD-AISC. This expected due to the fact that both LRFD-AISC and BS5950 uses the limit state concept in the design of steel frames while ASD-AISC is based on the allowable stress design. In the limit state design concept steel structure is designed according to the strength, serviceability and other limit states at which the structure becomes unfit to be able to serve to the purpose for which it is constructed. These limit states are checked under the factored loads that are given in both design codes. On the other hand in the allowable stress design the loads are taken as service loads without any factoring and the stresses develop in members are checked against their allowable values. As a result of this only elastic behavior of a steel structure allowed in allowable stress design code and allowable stresses are obtained by dividing the yield stress of the steel material by a safety factor. It is apparent that a steel structure will not be in an unsafe condition even though stresses in some of its members exceed the allowable stress values because of the fact that allowable stresses are much lower than the yield stress of the steel material. On the other hand, in the limit state design concepts the service loads are increased by load factors and the stresses develop under these loads are allowed to reach to yield strength values of steel material. Consequently, the design based on the limit state design concepts yields a lighter structure due to the fact that it takes into account the realistic behavior of steel structures. It is worthwhile to state the fact that in this study only gravity loadings are considered, the other loading cases are not considered in the optimum design. The difference between the optimum designs obtained according to ASD and LRFD design codes may be less when all the loading cases are considered. However, it is known that design codes based on the limit state concepts results in lighter designs [31]. It is interesting to notice that the optimum design obtained by considering the design constraints from LRFD-AISC design code is less but not very much different than the one obtained considering the design constraints from

Table 10 The comparison of optimum designs produced according to ASD-AISC, LRFD- AISC and BS5950 design codes for 85-member unbraced planar steel frame.

Member group	BS5950		ASD-AISC		LRFD-AISC	
	Section Designation	Area, cm^2 (in^2)	Section Des-	Area, cm^2 (in^2)	Section Des-	Area, cm^2 (in^2)
1	457X152X52 UB	66.5 (10.30)	W12X72	136.13 (21.1)	W16X31	58.90 (9.13)
2	457X152X52 UB	66.5 (10.30)	W14X68	129.03 (20.0)	W16X40	75.80 (11.75)
3	457X152X52 UB	66.5 (10.30)	W16X67	127.09 (19.7)	W18X40	75.90 (11.76)
4	457X152X52 UB	66.5 (10.30)	W10X45	85.81 (13.3)	W21X44	83.70 (12.97)
5	457X152X60 UB	75.9 (11.76)	W14X68	129.03 (20.0)	W21X44	83.70 (12.97)
6	457X152X52 UB	66.5 (10.30)	W10X49	92.90 (14.4)	W16X45	86.00 (13.33)
7	457X152X52 UB	66.5 (10.30)	W12X45	85.16 (13.2)	W21X44	83.70 (12.97)
8	457X152X52 UB	66.5 (10.30)	W12X53	100.65 (15.6)	W21X44	83.70 (12.97)
9	203X203X60 UC	75.8 (11.75)	W14X132	250.32 (38.8)	W14X30	57.30 (8.88)
10	203X203X52 UC	66.4 (10.29)	W14X109	206.45 (32.0)	W14X30	57.30 (8.88)
11	254X254X73 UC	92.9 (14.40)	W12X96	181.94 (28.2)	W16X36	68.10 (10.56)
12	203X203X46 UC	58.8 (9.11)	W12X65	123.22 (19.1)	W16X31	58.90 (9.13)
13	254X254X73 UC	92.9 (14.40)	W10X49	92.90 (14.4)	W8X31	58.60 (9.08)
14	203X203X52 UC	66.4 (10.29)	W24X55	104.51 (16.2)	W14X34	64.50 (10.0)
15	305X305X97 UC	123.0 (19.07)	W21X44	83.87 (13.0)	W8X40	75.60 (11.72)
16	203X203X71 UC	91.1 (14.12)	W24X68	129.68 (20.1)	W16X40	75.80 (11.75)
17	305X305X118 UC	150.0 (23.25)	W24X55	104.51 (16.2)	W18X60	114.00 (17.67)
18	254X254X73 UC	92.9 (14.40)	W24X68	129.68 (20.1)	W14X43	81.40 (12.62)
19	305X305X118 UC	150.0 (23.25)	W24X62	117.42 (18.2)	W14X61	116.00 (17.98)
20	305X305X97 UC	123.0 (19.07)	W21X44	83.87 (13.0)	W14X48	91.10 (14.12)
21	356X368X153 UC	195.0 (30.22)	W24X68	129.68 (20.1)	W12X72	136.00 (21.08)
Weight	33011kg (72711.45 lb)		47472.66 kg (104659.29 lb)		32868.54 kg (72462.73 lb)	

BS5950 in spite of the fact that the drift limitations are H/400 in LRFD and H/300 in BS5950. The reason for this is that in the discrete set of steel sections of the optimum design due to BS5950 there are only 96 available British steel sections (64 Universal Beam sections and 32 Universal Column sections) while in the design due to LRFD there are 272 W-sections in the discrete list. Hence the optimum design that is based on LRFD specifications has larger design space to select from compare to the design space which makes use of British steel sections. This difference provides better selection possibilities to the algorithms based on LRFD.

6 Conclusions

Adaptive harmony search algorithm presented in this chapter is efficient and robust algorithm that can be employed with confidence in the optimum design of real size steel skeletal structures. In this technique the harmony search parameters are dynamically adjusted by the algorithm itself taking into account varying features of the design problem under consideration. The algorithm itself automatically changes the values of harmony considering rate (*hmcr*) and pitch adjustment rate (*par*) depending on the experience obtained through the design process. Hence, varying features of a design space are automatically accounted by the algorithm for establishing a tradeoff between explorative and exploitative search for the most successful optimization process. It is shown through the design examples considered in the optimum design of real size steel structures that the adaptive harmony search method demonstrates good performance compare to standard harmony search method. Inspection of the design history of 209-member industrial factory building clearly shows the better performance in the convergence rate of the adaptive harmony search method compare to standard harmony search method. The optimum designs of 568-member and 1860-member steel structures are obtained by the presented technique without any difficulty. Furthermore, comparison carried out among seven recently developed metaheuristic optimization techniques has shown that adaptive harmony search algorithm finds the second lightest frame among the minimum weights obtained by these seven metaheuristic algorithms considered in this study while the standard harmony search method attains the heaviest design. Finally, the adaptive harmony search algorithm eliminates the necessity of carrying out a sensitivity analysis with different values of harmony search parameters whenever a new design problem is to be undertaken. This makes the algorithm more general and applicable to the optimum design of large size real-world steel structures. It is also shown in the last design example that use of different design codes results in different optimum designs. The allowable stress design method naturally yields heavier design due to the fact that nowhere in the frame stresses are allowed to reach their yield values. The load and resistance factor design and British Standards 5950 which are based on the ultimate state design concept gives lighter optimum designs as expected.

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