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# Optimum design of steel sway frames to BS5950 using harmony search algorithm

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**Abstract**

Harmony search method based optimum design algorithm is presented for the steel sway frames. The harmony search method is a numerical optimization technique developed recently that imitates the musical performance process which takes place when a musician searches for a better state of harmony. Jazz improvisation seeks to find musically pleasing harmony similar to the optimum design process which seeks to find the optimum solution. The optimum design algorithm developed imposes the behavioral and performance constraints in accordance with BS5950. The member grouping is allowed so that the same section can be adopted for each group. The combined strength constraints considered for a beam–column take into account the lateral torsional buckling of the member. The algorithm presented selects the appropriate sections for beams and columns of the steel frame from the list of 64 Universal Beam sections and 32 Universal Column sections of the British Code. This selection is carried out so that the design limitations are satisfied and the weight of steel frame is the minimum. The number of design examples considered to demonstrate the efficiency of the algorithm is presented.


*Keywords:* Optimum structural design; Combinatorial optimization; Stochastic search technique; Harmony search algorithm; Minimum weight; Steel frame

## 1. Introduction

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. This selection should be carried out in such a way that the steel frame has the minimum weight or cost while the behavior and performance of the structure is within the limitations described by the code of practice. Such problems fall into the subject of discrete optimization in which finding the optimum solution is a difficult task. Early optimum design algorithms based on a wide range of powerful mathematical programming methods have failed to satisfy the needs of practicing engineers. One of the reasons for this was that most of the mathematical programming techniques developed are based on the assumption of continuous design variables while in reality most of the structural optimization design variables are discrete in nature. Although some mathematical programming based methods have been developed for discrete optimum design problems they are not very efficient for obtaining the optimum solution of the large size practical design problems [1,2].

In recent years, structural optimization witnessed the emergence of novel and innovative design 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. The basic idea behind these techniques is to simulate the natural phenomena such as survival of the fittest, immune system, swarm intelligence and the cooling process of molten metals into a numerical algorithm. These methods are non-traditional search and optimization methods and they are very suitable and powerful in obtaining the solution of combinatorial optimization problems [3–22]. They do not require the derivatives of the objective function and constraints and they use probabilistic transition rules not deterministic rules. A large number of optimum structural design algorithms have been developed in recent years which are based on these effective, powerful and novel techniques [22–42].

One recent addition to these techniques is the harmony search algorithm [41–47]. This approach is based on the musical performance process that takes place when a musician

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searches for a better state of harmony. Jazz improvisation seeks to find musically pleasing harmony similar to the optimum design process which seeks to find optimum solution. The pitch of each musical instrument determines the aesthetic quality, just as the objective function value is determined by the set of values assigned to each decision variable. In this study optimum design algorithm based on harmony search method is developed that determines the optimum sectional designations of beams and columns of a steel frames from the Universal Beam and Universal Column section list of the British Code of BS5950 [48,49].

## 2. Discrete optimum design of steel frames to BS5950

The design of unbraced steel frames necessitates the selection of steel sections for its columns and beams from standard steel section tables so that the frame satisfies the serviceability and strength requirements specified by the code of practice while the economy is observed in the overall or material cost of the frame. When the design constraints are implemented from BS5950 [48] in the formulation of the design problem the following discrete programming problem is obtained.

$$\text{Minimize} = \sum_{r=1}^{ng} m_r \sum_{s=1}^{t_r} \ell_s \tag{1a}$$

Subject to $(\delta_j - \delta_{j-1})/h_j \le \delta_{ju}$

$$j = 1, \ldots, ns \tag{1b}$$

$$\delta_i \le \delta_{iu}, \quad i = 1, \ldots, nd \tag{1c}$$

$$\frac{F_k}{A_{gk} Py} + \frac{M_{xk}}{M_{cxk}} \le 1 \tag{1d}$$

or $k = 1, \ldots, nc$

$$\frac{F_k}{A_{gk} P_{ck}} + \frac{m_k M_{xk}}{M_{bk}} \le 1 \tag{1e}$$

$$M_{xn} \le M_{cxn} \quad n = 1, \ldots, nb \tag{1f}$$

$$B_{sc} \le B_{sb} \quad s = 1, \ldots, nu \tag{1g}$$

$$D_s \le D_{s-1} \tag{1h}$$

$$m_s \le m_{s-1} \tag{1i}$$

where Eq. (1a) defines the weight of the frame, $m_r$ is the unit weight of the steel section selected from the standard steel section table that is to be adopted for group $r$. $t_r$ is the total number of members in group $r$ and $ng$ is the total number of groups in the frame. $l_s$ is the length of member $s$ which belongs to group $r$.

Eq. (1b) represents the inter-storey drift of the multi-storey frame. $\delta_j$ and $\delta_{j-1}$ are lateral deflections of two adjacent storey levels and $h_j$ is the storey height. $ns$ is the total number of storeys in the frame. Eq. (1c) 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. $\delta_{ju}$ is the allowable lateral displacement. BS5950 limits the horizontal deflection of columns due to unfactored imposed load and wind loads to height of column/300 in each storey of a building with more than one storey. $\delta_{iu}$ is the upper bound on the deflection of beams which is given as span/360 if they carry plaster or other brittle finish.

Eq. (1d) defines the local capacity check for beam–columns. $F_k$ and $M_{xk}$ are the applied axial load and moment about the major 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. $M_{cx}$ is the moment capacity about the major axis. $nc$ is the total number of beam–columns in the frame.

Eq. (1e) represents the simplified approach for the overall buckling check for beam–columns. $m$ is the equivalent uniform moment factor given in Table 18 of BS5950. $M_{bk}$ is the buckling resistance moment capacity for member $k$ about its major axis computed from clause 4.3.7 of the code. $p_{ck}$ is the compression strength obtained from the solution of the quadratic Perry–Robertson formula given in Appendix C.1 of BS5950. It is apparent that the computation of compressive strength of a compression member requires its effective length. This can be automated by using the Jackson and Moreland monograph for frame buckling [50]. The relationship for the effective length of a column in a swaying frame is given as:

$$\frac{(\gamma_1\gamma_2)(\pi/k)^2 - 36}{6(\gamma_1+\gamma_2)} = \frac{\pi/k}{\tan(\pi/k)} \tag{3}$$

where $k$ is the effective length factor and $\gamma_1$ and $\gamma_2$ are the relative stiffness ratios for the compression member which are given as:

$$\gamma_1 = \frac{\sum I_{c1}/\ell_{c1}}{\sum I_{b1}/\ell_{b1}} \quad \text{and} \quad \gamma_2 = \frac{\sum I_{c2}/\ell_{c2}}{\sum I_{b2}/\ell_{b2}}. \tag{4}$$

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 non-linear equation (3) for $k$ results in the effective length factor for the member under consideration. Eq. (3) has the following form for non-swaying frames.

$$\frac{\gamma_1\gamma_2}{4}\left(\frac{\pi}{k}\right)^2 + \left(\frac{\gamma_1+\gamma_2}{2}\right)\left(1 - \frac{\pi/k}{\tan(\pi/k)}\right) + \frac{2\tan(\pi/2k)}{\pi/k} = 1. \tag{5}$$

Eq. (1f) is required to be imposed for each beam in the frame to ensure that each beam has the adequate moment capacity to resist the applied moment. It is assumed that slabs in the building provide sufficient lateral restraint for the beams.

Eq. (1g) is included in the design problem to ensure that the flange width of the beam section at each beam–column connection of storey $s$ should be less than or equal to the flange width of column section.

Eqs. (1h) and (1i) are required to be included to make sure that the depth and the mass per meter of column section at storey $s$ at each beam–column connection are less than or equal to the width and mass of the column section at the lower storey $s-1$. $nu$ is the total number of these constraints.