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Effect of Random Number Sequences On The Optimum Design of Castellated Beams With Improved Harmony Search Method

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Abstract

Random number sequences on the optimum design of steel castellated beams have an important effect in the minimum weight design. In the present research, this effect is investigated using an optimum design algorithm which is based on a recently developed improved harmony search method (IHS). Harmony search optimizer is a simulator of musically pleasing that is used to realize the experience of a musician for searching pleasing harmony, which is the musically performance process based numerical optimization technique. The minimum weights of both beams are taken as the objective functions while the design constraints are respectively implemented from The Steel Construction Institute Publication Numbers 5. The design methods adopted in these publications are consistent with BS5950 parts. The formulation of the design problem considering the limitations of the above mentioned turns out to be a discrete programming problem. The design algorithms based on the technique select the optimum Universal Beam sections, dimensional properties of hexagonal holes and total number of openings along the beam as design variables.

Keywords: structural optimization, catellated beams, metaheuristic search techniques, harmony search algorithm, random number sequences.

1. Introduction

Common steel I-beam sections can be modified to intensify their strength by creating an open-web section from a root beam. This is achieved by cutting the web of the root beam in a certain pattern and then re-welding the two halves to each other. As a result of this cutting and re-welding process the overall beam depth increases that causes increase in the capacity of section. There are mainly two types of open web-expanded beams: beams with circular openings referred to as cellular beams [1-3] and beams with hexagonal openings, also called as castellated beams. Since the Second World War the high strength to weight ratio of steel castellated beams has been a desirable item to structural engineers in their efforts to design even lighter and more cost efficient steel structures [4-6].

Castellated beams are steel sections with hexagonal openings that are made by cutting a saw tooth pattern along its centerline in the web of a rolled I beam section the length of the span. The two parts are then welded together to produce a beam of greater depth with halves of hexagonal holes in the steel section as shown in Figure 1. This hexagonal opening up of the original rolled beam increases the overall beam depth, moment of inertia and section modulus, while reducing the overall weight of the beam.

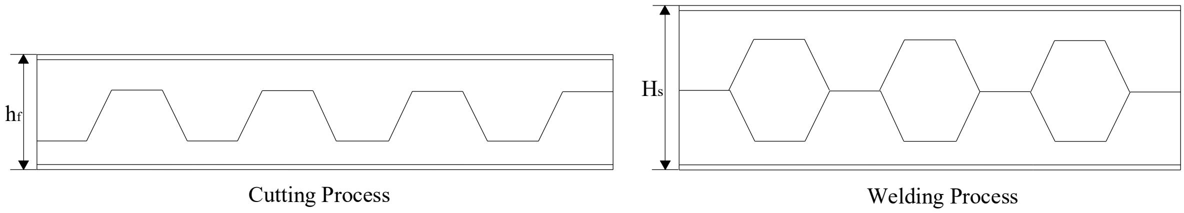

Fig. 1. Basic Process of Castellated Beam

Castellated beams have been used in various types of constructions for many years. The most common building types for these beams are office buildings, car parks, shopping centres and any structure with a suspended floor. Castellated beams provide a very economical solution for producing tapered members, which have been used extensively in big sports stadiums. They can also be used as gable columns and wind-posts. Castellated beams invariably produce a more efficient and economical solution than the original beams due to their weight and strength. Although the profile for any original I-beam section is standard or fixed, the major dimensions that are exact finished depth, hole dimensions and spacing of castellated beams are flexible. They are approximately 50% deeper and 50% stronger than the original member while without increasing the overall weight.

2. Optimum Design Problem of Castellated Beams

The strength of a beam with various web opening shall be determined based on the interaction of flexure and shear at the opening. Design constraints include the displacement limitations, overall beam flexural capacity, beam shear capacity, overall beam buckling strength, web post flexure and buckling, Vierendeel bending of upper and lower tees, local buckling of compression flange and practical restrictions between hole dimensions and the spacing between openings. The design procedure given here is taken from "The Steel Construction Institute Publication No: 005 titled "Design of Castellated Beams" [6]. The design methods are consistent with BS5950 part 1 and 3, and BS449 [7].

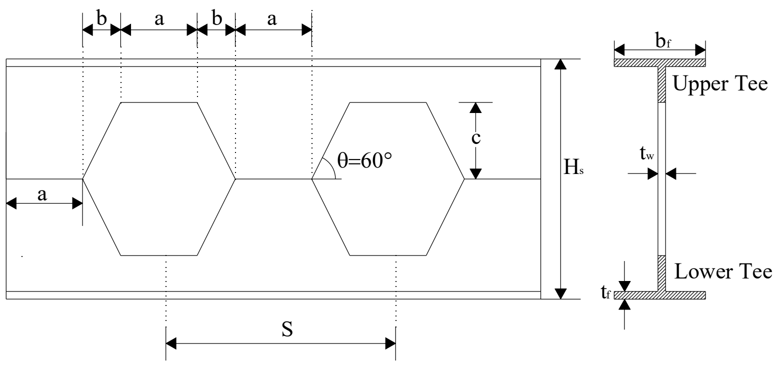

Fig. 2. Geometry and notation for cstellated beam

The standart profile geometry and notations used for castellated beams are shown in Figure 2. The dimensions of a castellated beam are described as following equations.

$$a = 0.5 \times (S - 2 \times c \times \cot 60^o) \tag{1}$$

$$b = c \times \cot 60^o \tag{2}$$

$$S = 2 \times (a + b) \tag{3}$$

$$H_S = h_f + c \tag{4}$$

Where, S is spacing between centers of the holes, h_f is the depth of original section, H_S is the final depth of the castellated beam, a, b and c are the dimensions of the hexagonal holes. Design properties and dimensions of the castellated beam are considered as constraints.

2.1. Maximum Stress Capacity

In the elastic design method the maximum stress can be expressed as following equations. Under applied load combinations maximum stress ($\sigma_{\max}$) in a castellated beam should not exceed an allowable stress capacity (σ_{allow}) to be able to resist the external loading.

$$K_1 = 1/(A_{tee} \times h_t) \quad (5)$$

$$K_2 = a/(4 \times Z_{tee}) \quad (6)$$

$$\sigma_{\max} = (K_1 \times M + K_2 \times V) \quad (7)$$

$$\sigma_{\max} \leq \sigma_{allow} \quad (8)$$

Where, A_{tee} is area of tee, h_t is distance between centroids of top and bottom tees, Z_{tee} is section modulus of tee, K_1 and K_2 are coefficients about beam properties. Stresses owing to bending and shear are shown in Figure 3.

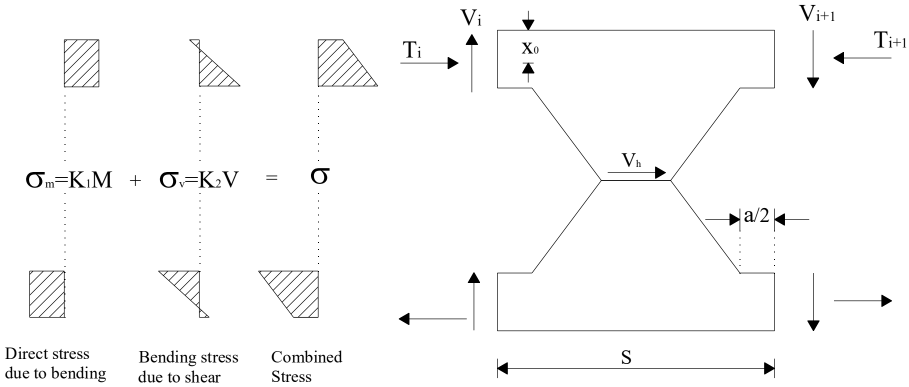

Fig. 3. Stress in tees of the castellated beam

2.2. Beam Shear Capacity

It is necessary to check two shear failure modes in castellated beams. The first one is the vertical shear capacity check of the beam. The sum of the shear capacities of the upper and lower tees gives the vertical shear capacity of the beam. The factored shear force in the beam should not exceed P_{vy}:

$$P_{vy} = 0.6 \times p_y \times (0.9 \times Minimum \ \ Area \ \ of \ \ web \ \ post) \quad (9)$$

The other is the horizontal shear check. The horizontal shear is developed in the web post due the change in axial forces in the tee as shown in Figure 3. The horizontal shear capacity in the web post of beam should not exceed P_{vh} where:

$$P_{vh} = 0.6 \times p_y \times (0.9 \times Area \ \ of \ \ webs \ \ upper \ \ and \ \ lower \ \ tees) \quad (10)$$

Neglecting the effect of the applied load and considering the vertical equilibrium and the rate of the variation of bending moment, then the following can be written.

$$V_{i+1} = V_i \quad (11)$$

$$M_i = T_i \times (H_S - 2x_0) \tag{12}$$

$$V_{i+1} = \frac{dM}{dx} = \frac{M_{i+1} - M_i}{S} = (T_{i+1} - T_i) \times \frac{(H_S - 2x_0)}{S} \tag{13}$$

For horizontal equilibrium:

$$V_h = T_{i+1} - T_i = V_{i+1} \frac{S}{H_S - 2x_0} \tag{14}$$

Where V is shear force, T is axial force and M is bending moment at the cross section of the castellated beam, S is distance between hexagonal hole centers and x_0 is the distance between the axial force to flange. These are all shown in Figure 3.

2.3. Web Buckling Capacity of Beam

In this study the compression flange of the castellated beam is assumed to be sufficiently restrained through the floor system it is attached to. Hence the overall buckling strength of the castellated beam is omitted. Experimental tests on castellated beams have shown that the web post flexural and buckling capacity is checked using the following equations according to BS5950 method.

$$\lambda_r = \frac{(H_s - 2 \times t_f) \times \sqrt{3}}{t_w} \tag{15}$$

$$P_w = H_s \times t_w \times P_c \tag{16}$$

$$V_{\max} \le P_w \tag{17}$$

In these equations; λ_r is slenderness ratio of web and H_s is overall depth of castellated beam. P_c value obtain from table 27(c) in BS 5950 according to λ_r and P_y values.

2.4. Vierendeel bending of upper and lower tees

The flexural capacity of the upper and lower tees under Vierendeel bending is critical. The transfer of shear forces across a single opening causes secondary bending stresses. The Vierendeel bending stresses around the opening may be calculated using interaction curves. For a symmetrical section, the shear force is resisted by the upper and lower web sections in proportion to their depth squared. Therefore, the shear force is divided equally between upper and lower web sections. The interaction between Vierendeel bending moment and axial force for the critical section in the tee should be checked as follows:

$$P_u = \frac{\sigma_{allow}}{K_2} \tag{18}$$

$$M_u = \frac{\sigma_{allow}}{K_1} \tag{19}$$

$$\frac{P_o}{P_u} + \frac{M}{M_p} \le 1.0 \tag{20}$$

Where P_o and M are the force and moment on the section due to external loading respectively. P_u is the maximum allowable shear force and M_p is the maximum allowable bending moment in the castellated steel beam.

2.5. Classification of Castellated Beam

The computation of the nominal moment strength M_p of a laterally supported beam necessitates first the classification of the castellated beam. The beam can be plastic, compact, non-compact or slender. In compact sections, local buckling of the compression flange and the web does not occur before the plastic hinge develops in the cross section. On the other hand in compact sections, the local buckling of compression flange or web may occur after the first yield is reacted at the outer fiber of the flanges. Classification I-shaped sections are carried out according to Table 1 that is given in BS5950 [10].

Table 1. Limiting width to thickness ratios

Type of Element	Plastic	Compact	Semi-compact
Outstand Element of Compression Flange	$\frac{b_f}{2t_f} \leq 8.5 \in$	$\frac{b_f}{2t_f} \leq 9.5 \in$	$\frac{b_f}{2t_f} \leq 15 \in$
For web, with neutral axis at mid-depth	$\frac{H_S - 2t_f}{t_w} \leq 79 \in$	$\frac{H_S - 2t_f}{t_w} \leq 98 \in$	$\frac{H_S - 2t_f}{t_w} \leq 120 \in$

The moment capacity is calculated as $M_p = P_y \times S$ for plastic or compact sections and as $M_p = P_y \times Z$ for semi-compact sections where $\varepsilon = (275/P_y)^{1/2}$ is constant, $\lambda_f = b_f/(2t_f)$ for I-shaped member flanges and the thickness in which b_f and t_f are the width and the thickness of the flange in which S is the plastic modulus and Z is the elastic modulus of section about relevant axis. P_y is the design strength of steel. $\lambda_w = h/t_w$ for beam web, in which $h = H_s - 2t_f$ plus allowance for undersize inside fillet at compression flange for rolled I-shaped sections. H_s is the overall depth of the section and t_w is the web thickness. h/t_w values are readily available in UB-section properties table.

2.6. Deflection of Castellated Beam

The limiting values for deflection of a beam under applied load combinations are given in BS5950, Part 1. According to these limitations the maximum deflection of a castellated beam should not exceed span/360. The deflection of castellated beam is computed using the virtual work method which is explained in detail in [9]. Figure 3 shows points of inflection at sections i and i+1. Shear force under applied load combination is distributed equally tees, the axial and horizontal forces in the upper and lower tees;

$$N_i = \frac{M_i}{h} \quad \text{and} \quad T_i = \frac{S(V_i + V_{i+1})}{2h} \tag{21}$$

Where; h is distance between the centre of upper and lower tees and S is distance between centrals of holes. The deflection at each point is found by applying a unit load at that point. Internal forces under a unit load are given by $\overline{V_i}$, $\overline{N_i}$, $\overline{T_i}$.

Deflection due to bending moment in tee; $y_{mt} = \frac{a^3}{24EI_T}(V_i\overline{V_i})$ (22)

Deflection due to bending moment in web post of beam;

$$y_{wp} = \frac{3 \times c^3}{Eb^3 t_w} \left[\log_e \left(\frac{a+2b}{a} \right) + \left(\frac{2a)}{a+2b} \right) - \frac{1}{2} \left(\frac{(a/2)^2}{((a/2)+b)^2} \right) - \frac{3}{2} \right] T_i \overline{T_i} \tag{23}$$

Deflection due to axial force in tee;

$$y_{at} = \frac{2S}{EA_T} (N_i \overline{N_i}) \tag{24}$$

Deflection due to shear in tee;

$$y_t = \frac{(a/2)}{GA_{TWEB}} (V_i \overline{V_i}) \tag{25}$$

Deflection due to shear in web post;

$$y_w = \frac{c}{Gbt_w} X \log_e \left(\frac{a+2b}{a} \right) T_i \overline{T_i} \tag{26}$$

Where E is elasticity modulus of steel, I_T is total moment of inertia of beam, G is shear modulus of steel and X is the web post form factor. The total deflection of a single opening under applied load is obtained by summing the deflections computed above.

$$y_T = y_{mt} + y_{wp} + y_{at} + y_t + y_w \tag{27}$$

On the other hand the deflection of the castellated beam is calculated by multiplying the deflection of each opening by the total number of openings in the beam as given in [7].

3. Improved Harmony Search Method

Harmony search (HS) algorithm is one of the recent editions to such stochastic search techniques founded on musically pleasing simulation to solve combinatorial optimization problems. This approach utilizes the experience of a musician for searching pleasing harmony similar to the optimum design process which seeks to find optimum solution. The pitch of each instrument determines the aesthetic quality; in just the same way as the objective function value is determined by the set of values assigned to each decision variable. Although HS method has been successfully applied to different practical optimization problems since its origination, the applications of the method in structural optimization are still immature and require a substantial amount of further research. Up until this time only a limited number of publications in the literature are carried out where the application of the technique in different problem areas encountered in the field. Amongst these restricted studies that look at the effectiveness of the HS method, Lee and Geem used the technique for minimum weight design of planar and space truss structures [8]. In 2009, Saka et al. and Değertekin focused to examine the optimum design of steel frames formulated according to BS5950 and LRFD-AISC design codes with HS, respectively [9-10]. Later, the success of the method in optimum W-sections for the transverse and longitudinal beams of grillage systems was investigated in Erdal and Saka [11-12]. Mainly small scale applications that consist of a small number of design variables were used in these aforementioned studies and all of them were concluded that HS algorithm was a very rapid and effective method for optimum design of such systems. Conversely, Hasançebi et al. evinced a comprehensive performance evaluation of the technique in the optimum design of real size trusses and frames where the design problem was formulated according to ASD-AISC in evinced a completely opposite outlook [13-14]. In comparison to those of other metaheuristic techniques, the performance of HS algorithm was qualified substandard with its slow convergence rate and unreliable search efficiency. An improvement of the technique was recommended for its application to new structural optimization problems, which in fact led to the motivation of the present study.

In the classical HS method the parameters harmony memory considering rate (η) and pitch adjusting rate (ρ) are selected prior to the application of the method and they are kept constant until the end of the iterations. The numerical applications have shown that the selection of values for η and ρ is problem dependent and the initial values selected affect the performance of the algorithm. Consequently, in order to determine the appropriate values of the harmony search parameters it is necessary to solve the optimization problem several times with different values and select the solution with minimum weight. It is apparent that such application devaluates the efficiency of the algorithm. In order to overcome this discrepancy, numbers of improvements are suggested in the literature. First, Mahdavi et al. have proposed an improved harmony search algorithm that uses variable ρ and bw in improvisation step where bw is an arbitrary distance bandwidth [15]. Later, Taherinejad has proposed a new function which could help the algorithm to explore vast search space while focusing well on local and global optimums [16]. And then, Hasançebi et al. suggested adaptive harmony search method where η and ρ are adjusted by the algorithm itself automatically using probabilistic sampling of control parameters [17]. Hence the algorithm tunes these parameters to advantageous values online during search. Eventually, Carbas and Saka have used the improved version of algorithm for latticed steel domes and some engineering problems, respectively. In the present study, different strategies are proposed for η and ρ to compare the minimum weight design of steel castellated beams [18]. ρ is updated using the concept suggested by Coelho and Bernert [19]. Before initiating the design process, a set of steel beam sections selected from an available UB 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. During optimization process selection of sections for design variables is carried out using these numbers. The basic components of the improved harmony search algorithm can now be outlined as follows.

3.1. Initialization of a parameter set

First a harmony search related optimization parameter set is specified. This parameter set consists of four entities known as a harmony memory size (μ), a harmony memory considering rate (η), a pitch adjusting rate (ρ) and a maximum search number (N_S). Out of these four parameters, η and ρ are dynamic parameters that vary from one solution vector to another, and are set to initial values of $\eta^{(0)}$ and $\rho^{(0)}$ for all the solution vectors in the initial harmony memory matrix. It is worthwhile to mention that in the standard harmony search algorithm these parameters are treated as static quantities, and hence they are assigned to suitable values chosen within their recommended ranges of $\eta \in [0.70, 0.95]$ and $\rho \in [0.20, 0.50]$.

3.2. Initialization of harmony memory matrix

Harmony memory matrix **H** is generated randomly initialized next. This matrix represents a design population for the solution of a problem under consideration, and incorporates a specified number of solutions referred to as harmony size (μ). Each solution vector ($\mathbf{I}^i$) consists of N_d design variables integer number between 1 to N_S (number of values) selected randomly each of which corresponds sequence number of design variables in the design pool, and is represented in a separate row of the matrix; consequently the size of $\mathbf{H}$ is ($\mu \times N_d$). I_i^j is the sequence number of the i^{th} design variable in the j^{th} randomly selected feasible solution.

$$\mathbf{H} = \begin{bmatrix} I_1^1 & I_2^1 & \dots & I_{N_d}^1 \\ I_1^2 & I_2^2 & \dots & I_{N_d}^2 \\ \dots & \dots & \dots & \dots \\ I_1^\mu & I_2^\mu & \dots & I_{N_d}^\mu \end{bmatrix} \begin{matrix} \phi(\mathbf{I}^1) \\ \phi(\mathbf{I}^2) \\ \dots \\ \phi(\mathbf{I}^\mu) \end{matrix} \tag{28}$$

3.3. Evaluation of harmony memory matrix

(μ) solutions shown in Eq. (42) are then analyzed, and their objective function values are calculated. The solutions evaluated are sorted in the matrix in the increasing order of objective function values, that is $\phi(\mathbf{I}^1) \leq \phi(\mathbf{I}^2) \leq \ldots \leq \phi(\mathbf{I}^\mu)$.

3.4. Improvising a new harmony

Upon sampling of a new set of values for parameters, the new solution vector $\mathbf{I}' = [I_1', I_2', .., I_{nv}']$ is generated. In the harmony memory consideration, each design variable is selected at random from either $\mathbf{H}$ or the entire discrete set. The probability that a design variable is selected from the harmony memory is controlled by a parameter called harmony memory considering rate (η). To execute this probability, a random number r_i is generated between 0 and 1 for each variable I_i. If r_i is smaller than or equal to η, the variable is chosen from harmony memory in which case it is assigned any value from the *i*-th column of the **H**, representing the value set of variable in μ solutions of the matrix (Eq. 29). If $r_i > \eta$, a random value is assigned to the variable from the entire discrete set.

$$I_i' = \begin{cases} I_i' \in \{I_i^1, I_i^2, ..., I_i^\mu\} & if\ r_i \leq \eta \\ I_i' \in \{1, .., N_S\} & if\ r_i > \mu \end{cases} \tag{29}$$

If a design variable attains its value from harmony memory, it is checked whether this value should be pitch-adjusted or not. Pith adjustment simply means sampling the variable's one of the neighboring values, obtained by adding or subtracting one from its current value. Similar to η parameter, it is operated with a probability known as pitch adjustment rate (ρ).

$$I_i'' = \begin{cases} I_i' \pm 1 & if\ r_i \leq \rho \\ I_i' & if\ r_i > \rho \end{cases} \tag{30}$$

3.4.1 Updating parameters

$$\rho_{(I)} = \rho_{(MIN)} + (\rho_{(MAX)} - \rho_{(MIN)}) \times Deg_{(I)} \tag{31}$$

where, $\rho_{(I)}$ is the pitch adjusting rate for generation I, $\rho_{(MIN)}$ is the minimum adjusting rate, $\rho_{(MAX)}$ is the maximum adjusting rate, and *i* is the generation number. $Deg_{(I)}$ is updated according to the following expression:

$$Deg_{(I)} = \frac{(HCOST_{MAX(I)} - HCOST_{MEAN})}{(HCOST_{MAX(I)} - HCOST_{MIN(I)})} \tag{32}$$

where, $HCOST_{MAX(I)}$ and $HCOST_{MIN(I)}$ are the maximum and minimum function objective values in generation I, respectively; $HCOST_{MEAN}$ is the mean of objective function value of the harmony memory matrix. The improvisation of η is carried out using the following expression;

$$\eta_{(I)} = \eta_{(MAX)} - (\eta_{(MAX)} - \eta_{(MIN)}) \times Deg_{(I)} \tag{33}$$

where, $\eta_{(I)}$ is the harmony memory considering rate for generation I, $\eta_{(MAX)}$ is the maximum considering rate, $\eta_{(MIN)}$ is the minimum considering rate, and I is the generation number.

3.5. Adaptive constraint handling

Once the new vector is obtained using the above-mentioned rules, it is then checked whether it violates problem constraints. If the new vector is severely infeasible, it is discarded. If it is slightly infeasible, it is included in the harmony memory matrix. In this way the violated harmony vector which may be infeasible slightly in one or more constraints is used as a base in the pitch adjustment operation to provide a new vector that may be feasible. This is carried out by using larger error values initially for the acceptability of the new design vectors and then this value is adjusted during the design cycles according to the expression given below;

$$Er(i) = Er_{MAX} - \frac{(Er_{MAX} - Er_{MIN})}{\sqrt{N_S}} \times \sqrt{i} \tag{34}$$

where, $Er(i)$ is the error value in iteration i, Er_{MAX} and Er_{MIN} are the maximum and the minimum errors defined in the algorithm respectively, N_s is the maximum iteration number until which tolerance minimization procedure continues. Eq. 48 provides larger error values in the beginning of the design cycles and quite small error values towards the final design cycles. Hence when the maximum design cycles are reached the acceptable design vectors remain in the harmony memory matrix and the ones which do not satisfy one or more design constraints smaller than the error tolerance would be pushed out during the design iterations.

3.6. Update of Harmony Matrix

After generating the new vector, its objective function value is calculated. If this value is better than that of the worst harmony vector in the harmony memory, it is then included in the matrix while the worst one is discarded out of the matrix. The updated harmony memory matrix is then sorted in ascending order of the objective function value.

3.7. Termination

Steps 3 and 4 are repeated until a pre-assigned maximum number of cycles N_{cyc} is reached. The number is selected large enough such that within this number no further improvement is observed in the objective function.

4. Numerical Example

Harmony search method based optimum design algorithm presented in the previous sections is used to design two castellated beams. Among the steel sections list 64 UB sections starting from 254X102X28 UB to 914X419X388 UB are selected to constitute the discrete set of steel profiles from which the design algorithm selects the sectional designations for the castellated beams [23]. For the hole depths discrete set is prepared that has 421 values which starts from 180mm and goes up to 600mm with the increment of 1mm. Another discrete set is arranged for the number of holes that contains numbers starting from 2 to 50 with the increment of 1.

4.1. 5-m span intermediate steel beam

A simply supported beam shown in Figure 4 is selected as first structural design example in order to compare the minimum weight of optimally designed steel castellated beams. The beam has a span of 5 m and is subjected to 5 kN/m dead load including its own weight. Two concentrated live loads with 40 kN weight also act at the beam as shown in the same figure. The maximum displacement of the beam under these loads is restricted to 14 mm while other design constraints are implemented from BS5950 as explained in Section 1 and 3.

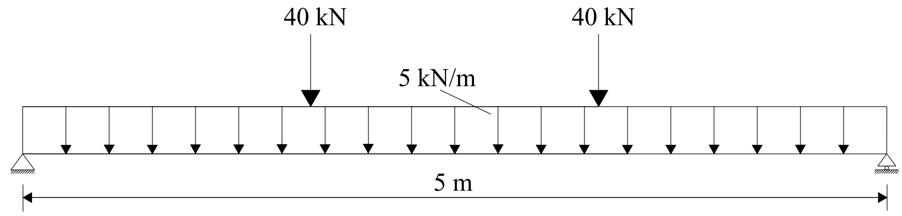

Fig. 4. Loading of 5-m simply supported beam

Considering the stochastic nature of HS technique, castellated beam with 5m span is designed with both improved and standard algorithms. The parameterization of the technique is conducted in line with the recommendations of the former studies [8-12], and thus the following parameter value set is used in solving the problem: a harmony memory size of $\mu = 50$, a maximum search number of $N_s = 5000$ are kept the same for both improved and standard HS algorithms. A harmony memory considering rate of $\eta = 0.90$, and a pitch adjusting rate of $\rho = 0.30$. It is important to note that these values of control parameters for η and ρ remain unchanged in the standard HS algorithm. Contrary to standard HS method, the values of η_{MAX} and ρ_{MAX} parameters in the IHS algorithm are taken as 0.99 and the 0.01 is assigned to η_{MIN} and ρ_{MIN}. These values are dynamically updated by the proposed algorithm during the optimization process.

The optimum results obtained by improved and standard versions of technique as well as the sectional designations and geometric dimensions for castellated beam are given in Table 2. It is apparent from the same table that improved HS has produced the lightest beam for steel castellated beam that has the minimum weight of 151.59 kg. The controlling interaction ratios of castellated beam are 0.99 for vierendeel bending, 0.49 for web-post buckling and 0.46 for horizontal shear. Classical HS algorithm has accomplished the heavier design with castellated beam which is 159.82 kg; 5.24% heavier for same 5m span.

Table 2. Optimum solutions of 5-m simply supported beam

	CASTELLATED BEAM				
	Section Design (UB)	Depth of Hole	Number of Holes	Max. Strength Ratio	Minimum Weight (kg)
IHS	254×146×31	218	14	0.99	**151.59**
HS	305×102×33	202	15	0.97	**159.82**

These results demonstrate that the proposed algorithm improves the performance of HS technique and it renders unnecessary the initial selection of the harmony search parameters. The design history curves for improved and standard versions of the technique for castellated beam is shown in Figure 5. This figure reveals the fact that IHS method has the faster convergence rate than classical HS algorithm.

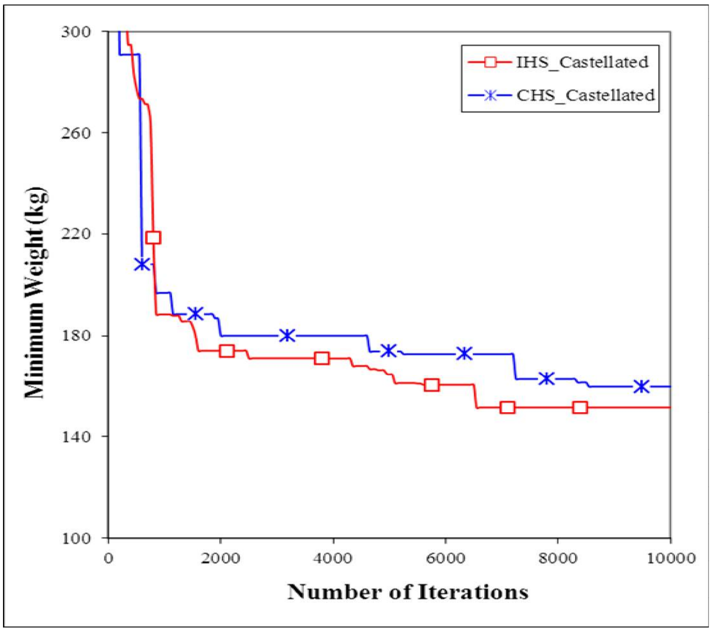

Fig. 5. Design history graph of 5-m simply supported beam

Within 5,000 analyses the proposed technique approaches a design in the vicinity of the optimum results. The maximum values of vierendeel bending moment ratio are 0.99 and 0.97 for castellated beam which are almost upper bound. This clearly reveals the fact that, in steel castellated beams, vierendeel bending moment constraints are dominant in the design problem. The IHS design algorithm presented selects 254×146×31 UB section for the castellated root beam shown in Table 2. The optimum castellated beam should be produced such that it should have 14 hexagonal holes each having 218 mm depth. The optimum shape of the castellated beam obtained from IHS method is demonstrated in Figure 6.

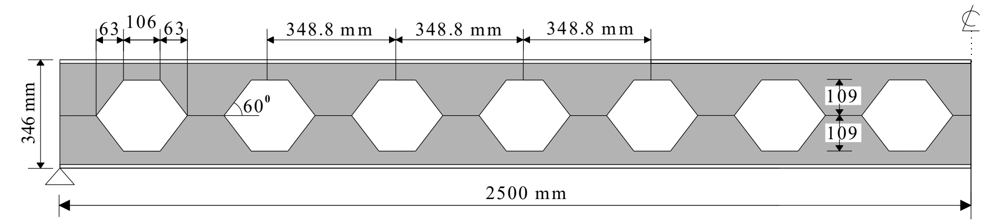

Fig. 6. Optimum profile section of the 5-m castellated beam

5. Effect of Random Number Sequences In IHS Algorithm

Since stochastic methods are based on eventual random decisions in operators, it is required to carry out a series of independent runs for castellated beam design example. Random number sequences always produce same number for different runs of the programs provided that the same seed value is used in each run. If the subroutine SEED is not called before the first call to subroutine RANDOM in FORTRAN, RANDOM always begins a seed value of one. However the use of different seed values in each run generates different random numbers. Since the IHS method also employs random number sequences in making decisions, the final result attained naturally is dependent upon the random numbers generated within each search.

To investigate the effect of random number sequences generated during the design procedure to the final result obtained by IHS technique, a design example for castellated beam is re-designed several times by using different seed value in each run. Firstly, 5-meter intermediate steel beam is optimized 50 times by running the program with different seed values. In the first run seed value of 1 is given in the beginning of the FORTRAN program, in the second run the seed value of 2 is assumed and in the 50th run the seed value of 50 is adopted. These runs are collected in two groups in order to investigate the effect of the initially selected maximum number of iterations in the IHS technique and variation of the seed value within that group of runs. In the first group of runs the maximum number of iterations is taken as 500 and seed values are changed from 1 to 50 in the each separate runs. In the second group of runs this number is taken as 5000. The minimum weights obtained in each run for the 5-meter intermediate steel beam are shown in Figure 7 and Figure 8 depending on the maximum number of iterations adopted in both group of run. It is apparent from the comparison of these two figures that the use of different seed values affects the minimum weight obtained in each run though some of the runs produce the same weight. But this effect becomes less if the maximum number of iterations in each run is choosen as a large number.

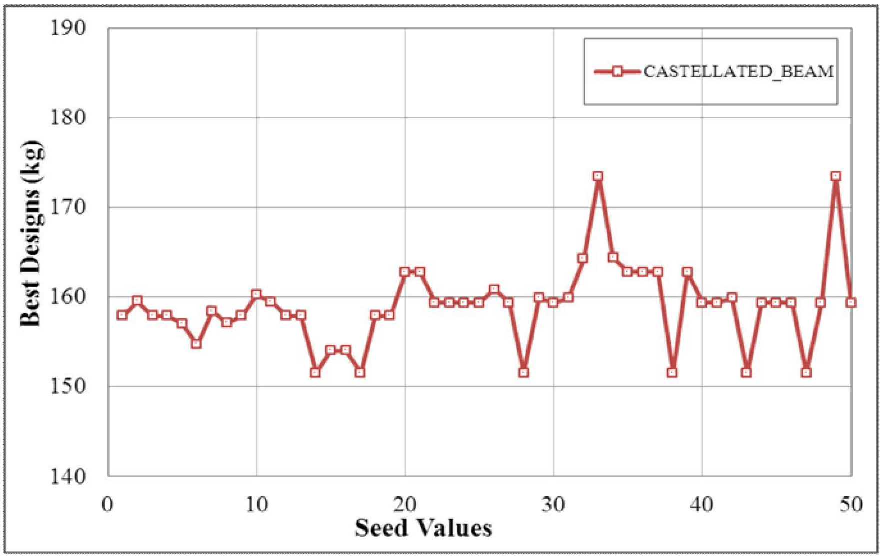

Fig. 7. Effect of Seed Values for 5-m span beam with 500 iterations

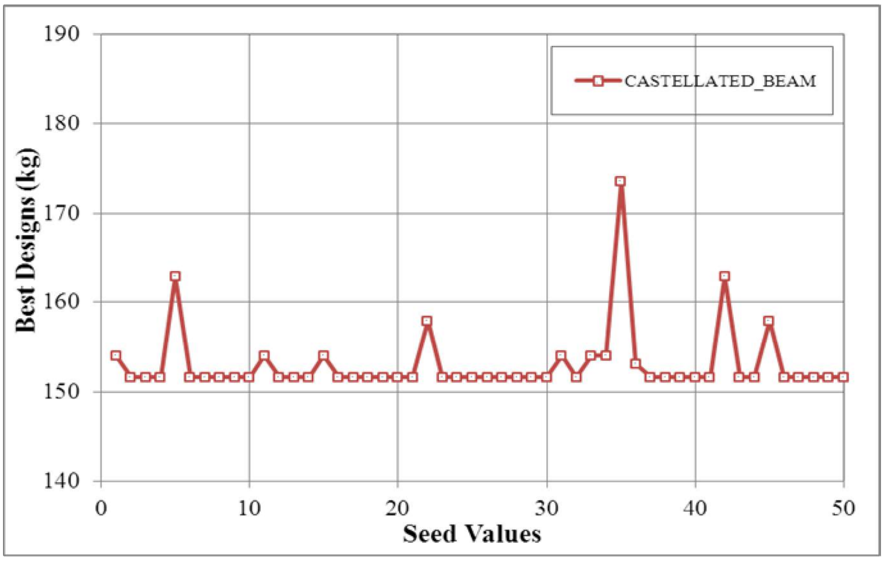

Fig. 8. Effect of Seed Values for 5-m span beam with 5000 iterations

6. Conclusions

The present research is the first study to cover a comparison of the performance of the adaptive and classical versions of harmony search algorithm during the optimization process of steel castellated beams. Unlike the classical algorithm where the update parameters, harmony memory consideration rate and pitch adjusting rate, of the technique are assigned to constant values throughout the search, the proposed algorithm benefits from updating these control parameters to advantageous values online during the iteration process. The efficiency of the improved harmony search algorithm in structural optimization is numerically examined using an example on size optimum design of castellated beams. The design history graph generated for the 5-meter simply supported beam problem using improved and classic harmony search algorithms clearly evince a significant performance improvement achieved with the former. Design history graphic for castellated beam reveals the fact that IHS method has the faster convergence rate than classical HS algorithm. Similarly, the effect of random number generation to the final result in the case of IHS algorithm is also investigated by running the optimum design program with different seed values. The minimum weights obtained in each run with different seed value for the steel castellated beams. It is apparent from the comparison of seed values with 500 and 5000 iterations that the use of different seed values affects the minimum weight obtained in each run though some of the runs produce the same minimum weight. However this effect becomes less if the maximum number of iterations in each run is selected as a large number.

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8. References

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