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Optimum design of semi-rigid connections using metamodels

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ABSTRACT

Considerable efforts were made in the past 15 years to develop strategies for the optimization of steel frames with semi-rigid connections, concentrating on the frames and not the connections, which were designed after the rest of the structure had been optimized. The analysis of semi-rigid connections requires the calculation of the moment-rotation curve (M_j-ϕ), which can be predicted using the Finite Element (FE) method. This is computationally expensive due to both the high number of degrees of freedom in the FE model and the nonlinear analysis required. In order to optimize such connections, a surrogate or metamodel of the FE model can be used. This paper puts forward a methodology for the optimal design of semi-rigid steel connections using metamodels generated with Kriging and Latin Hypercube, and optimized with the genetic algorithm method. This methodology was applied to two examples involving bolted extended end-plate connections, and was shown to work excellently at obtaining their optimal designs.

1. Introduction

Considerable efforts were made in the past 15 years to develop strategies for the optimization of steel frames with semi-rigid connections. This process involves minimizing the cost of the frames under specified design loads subject to stress and displacement constraints but with only the member sizes as design variables. The dimensions of the connection are not optimized, but a cost is given to them in the form of extra weight added to the steel members proportional to the rotational stiffness of that connection. Once the size of the structural profiles is obtained, what a designer currently does is to suggest an appropriate connection for the frame, which does not guarantee the resultant structure to be optimal.

Xu and Grierson [1] presented a computer-automated method to minimize the cost of the connections and members of steel frames. The cost of each member was represented by its weight, while the cost of each connection was related to its stiffness. Hayalioglu and Degertekin [2,3] presented a genetic algorithm (GA) based optimum design method for non-linear steel frames with semi-rigid connections and column bases. The design algorithm obtained the minimum total cost, using the objective function proposed by Xu and Grierson [1]. Simões [4] minimized the cost of the connections and members of the structure. He represented the cost of each member by its weight, while the cost of each connection was based on their rotation stiffness value converted into an equivalent structural weight. The weight minimization of only the structural profiles (members) of steel frames was carried out by Kameshki and Saka [5,6], Csébfalvi [7], and Liu [8]. Almusallam [9] and Al-Salloum and Almusallam [10] minimized the volume of the frame considering only the structural profiles. Pavlovčič et al. [11] presented a very detailed objective function that calculated the cost of the whole structure with rigid connections. It included all the essential fabrication costs, such as: welding, cutting, drilling, surface preparation, assembly, flange aligning, and painting, together with the steel and bolting material costs, transportation and erection costs. The cost for the connection was considered according to the beam and column dimensions and the bolt size was based on the full strength of the beam. The shop operation costs included: hole formation and welding of the stiffeners and the end-plate. The erection costs included bolting and site beam-to-column welding. Cabrero and Bayo [12] proposed a method for the optimum design of steel frames where the structural profiles and the values of stiffness and resistance for the connections were optimized. These connections were then dimensioned using these values.

In order to obtain the optimum design of steel frames, some researchers [1–12], use structural analysis to firstly obtain the rotational stiffness and moment resistance. These values are then used to determine the member sizes, after which, the connections of the structure may be optimized. Cho and Park [13] (referenced by Xu [14]) proposed an optimization model for the minimum cost design of: end-plate, bolted flange-plate and welded flange-plate beam-to-column connections. The design variables were: the number and size of the bolts, the dimensions of the end-plates, the thickness and length of the flange plates, the size and length of the welds, and the sizes of the cleats and seats. The cost function for the connection was defined by the cost of the: design variables, material, labor and fabrication.

In the case of the analysis of semi-rigid connections, there are many models to predict rotational behavior which is accounted for

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by the moment-rotation curve (M_j-ϕ). The most commonly used are: analytical, empirical, experimental, informational (metamodel), mechanical and numerical. A more detailed discussion of these is given in [15], where a review of the state-of-the-art of the modeling of the behavior of steel frame connections was carried out.

Numerical models based on the Finite Element Method (FEM) are currently one of the most widely used alternatives of obtaining the mechanical behavior of a connection. There are four reasons for this [16]: 1) As a means of overcoming the lack of experimental results; 2) To understand important local effects which are difficult to measure with sufficient accuracy; 3) To generate extensive parametric studies; and 4) To allow for the introduction into the model of: large deformations and displacements, plasticity, strain-hardening, instability effects, contacts between plates and pre-stressing of bolts.

The optimum design of semi-rigid connection using Finite Element Analysis (FEA) is computationally expensive [16–20] due to the high number of degrees of freedom (DOF) associated with both the dense Finite Element (FE) meshes and the nonlinear analysis required. One alternative to this is to use surrogate models or metamodels [17–20] developed from the FE model to evaluate the design space in search of the optimum. The most used methodologies to build metamodels are: Response Surface Methodology (RSM) [21,22] and Design and Analysis of Computer Experiments [23], which in the literature is known as the DACE methodology [24–26].

The use of metamodels to represent the behavior of semi-rigid connections has been carried out by several researchers, all of whom used Artificial Neural Networks (ANN). Yun et al. [27] characterized the cyclic behavior of connections from the results of structural testing. Jadid and Fairbairn [28] predicted the moment-curvature parameters from experimental data for beam-to-column connections. Anderson et al. [29] predicted the bilinear approximation of the moment-rotation curves of minor axis beam-to-column flush end-plate connections. Stavroulakis et al. [30] predicted the global moment-rotation curve for single web angle beam-to-column connections. De Lima et al. [31] predicted the flexural resistance and initial stiffness of beam-to-column steel connections. Guzelbey et al. [32] estimated the rotation capacity of wide flange beams. Pirmoz and Golizadeh [33] and Salajegheh et al. [34] estimated the behavior of bolted top-seat angle connections with web angles. Kim et al. [35] modeled the non-linear hysteretic cycle for bolted beam-to-column angle connections in steel frames.

Using the rotational stiffness and moment resistance results of optimized steel frames, this paper proposes a methodology for the optimum design of semi-rigid steel beam-to-column bolted extended end-plate connections using metamodels. These were built using a combination of Kriging [36,37] with Latin Hypercube Sampling (LHS) [38–41]. The data for generating the metamodels was produced using the full three-dimensional (3D) ANSYS FE model of Díaz et al. [16]. The metamodels were used to predict the objective function value (the connection cost) and the constraint values (the rotational stiffness and the moment resistance of the connection). The total cost of the connection was minimized using GA [42]. The methodology was implemented using the Matlab programming language [43] and applied to two connections to illustrate the proposed methodology. The results of these were compared with those of Cabrero and Bayo [12].

2. Definition of a bolted extended end-plate connection

The geometric parameters which define the semi-rigid steel beam-to-column bolted extended end-plate connection of Fig. 1 are given below [16]. The structure used to analyze the connection [16] is given in Fig. 2.

a. Beam: flange thickness (t_{fb}), flange width (b_{fb}), height (h_b), root radius (r_b), and web thickness (t_{wb}).
b. Bolt: distance between the tension rows (p_x), distance between the lower tension row and compression row (p), distance from upper tension row to top of end-plate (e_x), edge distance (e), distance between row 1 and the beam top flange (a_1), distance between row 2 and the beam top flange (a_2), distance between row 3 and the beam bottom flange (a_3), horizontal distance between bolt and beam web (m), gauge (w), hole clearance (d_0), and nominal bolt diameter (d_b). Note that the value of d_0 is dependent on d_b, Eq. (2).
c. Column: flange thickness (t_{fc}), flange width (b_{fc}), height (h_c), root radius (r_c), and web thickness (t_{wc}).
d. End-plate: distance of upper edge below beam top flange (l_{pu}), distance of lower edge below beam bottom flange (l_{pl}), height (h_{ep}), thickness (t_{ep}), and width (b_{ep}).
e. Load: stiffener thickness (t_s), which is considered equal to t_{fb}.
f. Weld throat thicknesses: beam flange and end-plate (a_f), beam web and end-plate (a_w).

2.1. Analysis of semi-rigid connection

Steel portal frames were traditionally designed assuming that beam-to-column connections are ideally pinned or fully rigid, whereas, due to the finite stiffness of the connections, the true behavior is somewhere between these two extremes. There is currently a great range of studies of steel frames with semi-rigid connections [44–49]. These studies agree that when analyzing a frame, the rotational behavior of the connections must be considered. The true behavior of a connection can be incorporated within the global analysis of the structure by using the moment-rotation curve (M_j-ϕ), (Fig. 3).

In this work, the 3D FE model of [16] was used to obtain the behavior of steel beam-to-column bolted extended end-plate connections. This is achieved by determining the mechanical properties of the connection in terms of its rotational stiffness (S_j), moment resistance ($M_{j,Rd}$), and rotational capacity (ϕ_{Cd}), (Fig. 3). The FE model includes contact and sliding between different components, bolt pre-tension, geometric and material nonlinearity.

3. Definition of the connection optimization problem

The standard formulation of an optimization problem with equality and inequality constraints is given by Eq. (1).

$$\begin{array}{lll} \text{Minimize} & f(\mathbf{x}) & \\ \text{Subject to} & h_j(\mathbf{x}) = 0 & (j = 1, \ldots, n_e) \\ & g_k(\mathbf{x}) \geq 0 & (k = 1, \ldots, n_i) \\ & x_i^l \leq x_i \leq x_i^u & (i = 1, \ldots, n_v) \end{array} \tag{1}$$

where: $\mathbf{x}$ is the vector of design variables, $f(\mathbf{x})$ is the objective function, $h_j(\mathbf{x})$ is the *jth* equality constraint, $g_k(\mathbf{x})$ is the *kth* inequality constraint, n_e is the number of equality constraints, n_i is the number of inequality constraints, x_i^l is the lower bound of the *ith* design variable, x_i^u is the upper bound of the *ith* design variable, and n_v is the total number of design variables.

3.1. Design variables

The six design variables used in the optimization are the:

1) Distance between the tension rows (p_x),
2) Distance from the upper tension row to the top of end-plate (e_x),
3) Edge distance (e),
4) Nominal bolt diameter (d_b),
5) Thickness of end-plate (t_{ep}), and
6) Width of end-plate (b_{ep}).

3.1.1. Dependent variables

From the twenty eight geometric parameters which define the extended end-plate connection of Fig. 1, the only six which are independent were selected as design variables. The remaining twenty two are determined in the following way: