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Optimal design of long-span steel portal frames using fabricated beams

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ABSTRACT

This paper considers the optimal design of fabricated steel beams for long-span portal frames. The design optimisation takes into account ultimate as well as serviceability limit states, adopting deflection limits recommended by the Steel Construction Institute (SCI). Results for three benchmark frames demonstrate the efficiency of the optimisation methodology. A genetic algorithm (GA) was used to optimise the dimensions of the plates used for the columns, rafters and haunches. Discrete decision variables were adopted for the thickness of the steel plates and continuous variables for the breadth and depth of the plates. Strategies were developed to enhance the performance of the GA including solution space reduction and a hybrid initial population half of which is derived using Latin hypercube sampling. The results show that the proposed GA-based optimisation model generates optimal and near-optimal solutions consistently. A parametric study is then conducted on frames of different spans. A significant variation in weight between fabricated and conventional hot-rolled steel portal frames is shown; for a 50 m span frame, a 14–19% saving in weight was achieved. Furthermore, since Universal Beam sections in the UK come from a discrete section library, the results could also provide overall dimensions of other beams that could be more efficient for portal frames. Eurocode 3 was used for illustrative purposes; any alternative code of practice may be used.

1. Introduction

In the UK, it is estimated that steel portal frames account for 90% of all single-storey buildings [1]. The vast majority of portal frames use hot-rolled steel sections for the column and rafter members. Using such sections, frames economically achieve spans of up to 50 m [2].

For longer span frames, an alternative to the use of hot-rolled steel sections could be fabricated steel beam sections [3,4]. Such fabricated beams, built-up through the welding of steel plates, have become increasingly popular for multi-storey buildings, where clear spans of up to 100 m are achievable [1]. In this paper, the use of such fabricated beams for portal frames will be considered using a genetic algorithm (GA) to size the dimensions of the fabricated beams.

Genetic algorithms have previously been applied to the design optimisation of hot-rolled steel portal frames [5–8]. In these studies, only four design variables were used; namely, the cross-section sizes of the columns and rafters, and the length and depth of the eave haunch [8]. The design used in this present paper was elastic. Phan et al. [8] showed that elastic design was sufficient since the design was controlled by deflection limits.

On the other hand, a design optimisation of fabricated steel sections can involve up to thirteen design variables (see Section 2.2); these being, the dimensions of the plates of each of the members as well as the dimensions of the haunch. To reduce the number of function evaluations, an effective means of enhancing the reliability is required.

Three benchmark frames are considered, with the frames designed elastically under gravity load in accordance with Eurocode 3. Both ultimate and serviceability limit states are considered. A parametric study is conducted to explore the full search space for single storey steel buildings. Spans of 14 m to 50 m and eave heights varying from 4 m to 12 m were considered.

2. Benchmark frames

Three frames are considered:

Frame A A span of 40 m and a height of 10 m.
Frame B A span of 50 m and a height of 12 m.
Frame C A span of 60 m and a height of 12 m.

The pitch and frame spacings for all three frames are 6° and 6 m, respectively; such pitch and frame spacings are typical for portal

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frames in the UK [2]. The column bases are assumed to be pinned. It is also assumed that the steel sections are fabricated from S275 steel [9].

2.1. Portal frames composed of universal beams

Frames are generated by selecting universal beam sections for the column and rafters from a list of 80 standard sections given in the SCI "Steel building design: Design data" book [10]. The column, rafter and haunch sections are considered as discrete variables with haunch length (H_L) treated as a continuous variable [8].

Four universal beam cases (UBC) are considered:

- UBC1 has two decision variables: the column and rafter sections. The haunch is assumed to be the same as the rafter section and the haunch length is fixed at 10% of the span.
- UBC2 has three decision variables: column, rafter and haunch sections. The haunch length is fixed at 10% of the span.
- UBC3 also has three decision variables: column section, rafter section and haunch length. The haunch is assumed to be the same as the rafter section.
- UBC4 has four decision variables: column section, rafter section, haunch section and haunch length.

2.2. Portal frames composed of fabricated beams

Portal frames composed of fabricated beams are generated with the dimensions described below. The plate thickness is treated as discrete variables and used for the web and flange. 34 plate thicknesses available within the UK are considered; at 1 mm spacings 6–25 mm; 5 mm spacings 30–80 mm and individually 12.5, 28 and 63.5 mm. The depths and breadths of the sections are treated as continuous variables with a range of 110 mm to 2000 mm and 50 mm to 600 mm, respectively.

Three Fabricated Beam Cases (FBC) are considered as follows with 13 decision variables in total: h_C, h_R, h_H, b_C, b_R, b_H, t_{wC}, t_{wR}, t_{wH}, t_{fC}, t_{fR}, t_{fH}, and H_L. The notations use standard (descriptive) terminology: h for height, b for breadth and t for thickness, including web (w in subscript) and flange (f in subscript) thicknesses; in subscript, C for column, R for rafter and H for haunch.

FBC1 is defined as follows: h_C, $h_R = h_H$, $b_C = b_R = b_H$, $t_{wC} = t_{wR} = t_{wH}$, $t_{fC} = t_{fR} = t_{fH}$, and H_L. FBC1 also has 13 decision variables that are further constrained as shown in the equations.

FBC2 has the following properties: h_C, $h_R = h_H$, b_C, $b_R = b_H$, t_{wC}, $t_{wR} = t_{wH}$, t_{fC}, $t_{fR} = t_{fH}$, and H_L. Using restrictions for FBC1 and FBC2 corresponds to the operational simplicity and possibly economy of using smaller numbers of plate sizes.

FBC3 has the following properties: h_C, h_R, h_H, b_C, b_R, b_H, t_{wC}, t_{wR}, t_{wH}, t_{fC}, t_{fR}, t_{fH}, and H_L.

In addition, discussions with manufacturers of fabricated beams suggest that the following geometric constraints are required in order to ensure that the plates can be welded and handled practically on the fabrication shop floor:

- $h_C > 3t_{fC}$; $h_R > 3t_{fR}$; $h_H > 3t_{fH}$.
- $t_{fC} > t_{wC}$; $t_{fR} > t_{wR}$; $t_{fH} > t_{wH}$.

3. Frame actions

In this paper, the permanent actions (G) and variable actions (Q) assumed to act on the frames are as follows:

G: 0.55 kN/m^2 + self-weight of primary steel members
Q: 0.60 kN/m^2.

Under vertical load, the frame should be verified at the ultimate and serviceability limits where the deflection limits and actions combination as recommended by the SCI [8,11] are adopted. Variable and permanent actions are factored in accordance with Eurocode 3: design of steel structures [12]:

$$\mathrm{ULS} = 1.35G + 1.5Q$$

$$\mathrm{SLS1} = 1.0G + 1.0Q \quad (\text{for absolute deflection})$$

$$\mathrm{SLS2} = 1.0Q \quad (\text{for differential deflection relative to adjacent frame})$$

where,

ULS is the ultimate limit state
SLS is the serviceability limit state.

4. Ultimate limit state design

4.1. Elastic frame analysis

Modern practice has shown that plastic design produces the most efficient designs in the majority of cases [2,13]. Elastic design is still used, particularly when serviceability limit state deflections will control frame design [14,15]. Phan et al. [8] have demonstrated that, if the SCI deflection limits are adopted, serviceability limit states control design. Therefore, elastic design is used in this paper.

A frame analysis programme, written by the authors in MATLAB, was used for the purpose of the elastic frame analysis. The internal forces, namely, axial forces, shear forces, and bending moments can be calculated at any point within the frame. It should be noted that second-order effects are not considered, since the geometry in the benchmark frames satisfy the requirements for in-plane stability of the sway check method, described in BS 5950 [16].

4.2. Ultimate limit state design requirements

Structural members are designed to satisfy the requirements for local capacity in accordance with Eurocode 3 [17]. Specifically, members are verified for capacity under shear, axial, and moment, and combined moment and axial forces. For fabricated beams, the buckling curves used are taken in accordance with the UK National Annex [18]. Sections are classified based on the axial and bending forces in conjunction with their geometric properties as class 1, 2 or 3. For class 1 or 2 sections, a plastic design approach is used in verification. For class 3 sections an elastic verification is substituted in the design. Sections outside this range (class 4) are excluded through use of a GA penalty.

Local buckling verifications are excluded under the proviso that a more detailed design of any necessary web and flange stiffeners will be conducted on the optimum selected sections. For example the stiffeners are generally required in the eave connections to allow for the concentrated axial forces transference from the rafter to the column.

4.2.1. Shear capacity

The shear force, V_{Ed}, should not be greater than the shear capacity, $V_{c,Rd}$.

$$V_{Ed} \le V_{c,Rd} \tag{1}$$