DEVELOPMENT OF COLD-FORMED STEEL MOMENT-RESISTING FRAMES USING OPTIMUM BEAMS IN SEISMIC APPLICATIONS

Seyed Mohammad Mojtabaei¹, Ioannis Papargyriou², Iman Hajirasouliha³, Jurgen Becque⁴, Kypros Pilakoutas⁵

ABSTRACT

Cold-Formed steel (CFS) elements are increasingly being used as the main load-carrying members in modern construction, including in seismic regions. The main deficiency of CFS elements is their susceptibility to premature buckling due to their limited thickness, which results in less seismic energy dissipation and lower ductility compared to their conventional hot-rolled counterparts. However, the seismic characteristics of CFS elements can be increased by using more efficient cross-sectional shapes. This paper aims to improve the seismic performance and post-buckling behaviour of CFS lipped channel sections by optimising their geometry to achieve maximum energy dissipation capacity under cyclic loads. A novel optimisation framework is presented, using the Particle Swarm Optimisation (PSO) method, linked to detailed non-linear finite element models in ABAQUS. The relative dimensions of the cross-section and the inclination of the lip stiffeners are considered as the main design variables. All Eurocode 3 plate slenderness limit values and limits on the relative dimensions of the cross-sectional components, as well as a range of practical and manufacturing limitations, are considered as design constraints in the optimisation problem. It is shown that the proposed optimisation method can significantly increase the energy dissipation capacity of the CFS sections. Subsequently, the assessment of CFS beams connections is conducted by simulations of cyclically loaded moment resisting connections using an experimentally validated model. Finally, the efficiency of the optimised sections is investigated at the structural level by conducting non-linear cyclic analyses on a CFS moment resisting portal frame with CFS back-to-back lipped channel sections for the columns and the beam. The results indicate that using the optimised CFS sections can considerably improve the seismic performance of the CFS frames, leading to higher energy dissipation capacity and lower global damage under strong earthquakes.

Keywords: Cold-formed steel sections (CFS); Particle Swarm Optimisation (PSO); Finite Element Modelling; Cyclic Analysis; Energy Dissipation Capacity

1. INTRODUCTION

Cold-formed steel (CFS) sections are manufactured by rolling or brake-pressing thin steel sheets into cross-sectional shapes at ambient temperature. Generally, CFS structures provide a wide range of advantages compared to their hot-rolled counterparts, most importantly a high strength-to-weight ratio, making efficient use of the material. Research and design experience related to the use of CFS members as load-bearing elements in the buildings has steadily grown over past decades and it has been shown

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that optimisation of CFS elements with respect to their maximum capacity in bending or compression can lead to significant material savings (Ma et al. 2015, Ye et al. 2016). However, there is limited available research on the energy dissipation capacity of individual CFS load-bearing elements and CFS moment-resisting frames. Calderoni et al. (2009), Padilla-Llano, Eatherton, & Moen (2016) and Mojtabaei et al. (2018) investigated the monotonic and cyclic performance of CFS channel beams, and the results showed a substantial energy dissipation capacity in the CFS sections. Experimental tests were conducted by Sabbagh et al. (2012) on the seismic behaviour of moment resisting connections, using channels with curved flanges (Figure 1). The obtained hysteretic behaviour of the CFS moment-resisting connections indicated adequate seismic energy dissipation capacity and acceptable ductility to satisfy code-requirements for seismic design. However, by optimising the cold-formed steel sections the seismic characteristics of CFS structures can be further improved, and additional material savings can be achieved.

In the research presented in this paper, the Particle Swarm Optimisation (PSO) algorithm developed by Ye (2016) was combined with the finite element program ABAQUS v6.14 (2014) to optimise CFS lipped channels with respect to their energy dissipation capacity under cyclic loading. The relative dimensions of the cross-sections and the inclination of the lip stiffeners were considered as the main design variables. The optimum lipped channel was then applied as a beam in a moment resisting connection shown in Figure 1 and previously used in tests by Sabbagh (2012). The model included material and geometrical nonlinearity. The hysteretic behaviour of a connection containing the optimum beam was then compared to the behaviour of an equivalent connection employing a standard beam. Finally, the seismic characteristics of CFS moment resisting frames were evaluated by representing the studied moment resisting connections by rotational springs.

2. OPTIMISATION OF SINGLE LIPPED CHANNEL BASED ON ENERGY DISSIPATION

Previous research studies have shown that Finite Element (FE) models can be used to accurately predict the load carrying capacity and post-buckling behaviour of CFS sections, provided that the appropriate element type, material parameters and imperfection profiles are selected (Yu and Schafer 2007, Becque and Rasmussen 2009a, 2009b, Ye 2016). In this paper, the general purpose FE package ABAQUS v6.14 (2014) was used. The modelling techniques used in the FE models were first verified against a series of tests on CFS back-to-back channels carried out by Ye (2016). The reader is referred to (Ye 2016) for the details of the validation procedure. The model was then used to predict the deformation behaviour of the prototype beams under cyclic loading and to search for the optimum cross-sectional shapes which maximize the energy dissipation.
2.1 FE modelling description and optimisation problem

An FE model of a 1250 mm long cantilever was created. For the sake of simplicity only half of the connection, containing one single channel was modelled. The cantilever was loaded by applying a vertical displacement at the end section, while twisting of the end section was restrained. The mesh size was maintained at 20×20 mm², identical to the one used in the validation process. The cantilever setup was devised to be representative of the portion of a beam between the connection and the point of inflection in a typical moment resisting frame.

A standard commercially available CFS lipped channel section is shown in Figure 2. The amount of material used in this ‘standard’ channel was kept constant in the optimization process (i.e. all channels had the same total developed plate width L=415 mm and the same thickness t=1.5 mm). The independent design variables are listed in Table 1 and consisted of cross-sectional dimensions and the angles included between various plate segments. All cross-sections were required to meet the dimensional restrictions determining the range of validity of the EN1993-1-3 specifications. These restrictions, listed in Table 1, pertain to plate slenderness limits, as well as limit values of the relative cross-sectional dimensions and were included as constraints in the optimisation problem. These restrictions were imposed in order to obtain a cross-section which can be validly designed using EC3. However, an important practical constraint was also imposed on all cross-sections: in order to allow the section to support a roof or floor diaphragm and be screw-connected to it, a minimum flat flange width of 50 mm was specified. The range of web height (200 ≤ h ≤ 400 mm) and the minimum dimension of lip stiffener (c≥10 mm) are selected after discussion with a CFS manufacturer company.

![Figure 2. Standard CFS beam cross-section](image)

<table>
<thead>
<tr>
<th>Prototype section</th>
<th>Design variables</th>
<th>Constraints based on EC3</th>
<th>Comments</th>
<th>Manufacturing &amp; practical limitations (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>x1=c/b</td>
<td>0.2≤c/b≤0.6</td>
<td>EN1993-1-3 Table 5.1 and Equation (5.2a), Clause 5.5.3.2(1)</td>
<td>200≤b≤400</td>
</tr>
<tr>
<td></td>
<td>x2=b/L</td>
<td>b/t≤60</td>
<td></td>
<td>b≥50</td>
</tr>
<tr>
<td></td>
<td>x3= θ1</td>
<td>c/t≤50</td>
<td></td>
<td>c≥10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>h/t≤500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>π/4≤θ1≤3/4π</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The yield stress $f_y$, the elastic modulus $E$ and the Poisson’s ratio of the material were taken as 450 MPa, 210 GPa and 0.3, respectively. The optimisation procedure aimed to optimise the CFS cross-section...
subject to the cyclic loading protocol described by Method B of ASTM-E2126-07 (Figure 3), with respect to its energy dissipation $E(X)$ over the cyclic loading history up to a drift ratio of 4%. This limit corresponds to the rotation capacity required for Special Moment Frames (SMF) according to AISC Seismic Provisions (2016) and allows for a consistent comparison.

The optimisation problem was formulated mathematically as follows:

$$\max E(X)$$

$$X_i^L \leq X_i \leq X_i^U \quad (i = 1, \ldots)$$

where $X$ denotes the vector containing the independent cross-sectional variables $X_i$ listed in Table 1. $X^U$ and $X^L$ indicate the upper and lower bounds, respectively, of the design variables which are subject to constraints.

![Figure 3. Cyclic loading regime (ASTM-E2126-07)](image)

**2.2 Optimisation results**

Figure 4 shows a typical convergence diagram and illustrates that there was no obvious increase of the objective value (i.e. the energy dissipation capacity) after about 100 iterations.

![Figure 4. Iteration history of optimization](image)
The hysteretic behaviour and the failure mode of the standard and optimum CFS lipped channel sections under cyclic loading are shown in Figures 5 and 6, respectively. Table 2 summarizes the cross-sectional dimensions obtained from the optimisation process and compares them with the standard lipped channel section with the same amount of material taken as a starting point.

![Figure 5. Cyclic behaviour of standard and optimum CFS lipped-channel sections](image1)

![Figure 6. Failure mode of (a) standard and (b) optimum CFS lipped-channel sections](image2)

<table>
<thead>
<tr>
<th>Beam type</th>
<th>Dimensions (mm)</th>
<th>Energy dissipation (J)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>H</td>
<td>b</td>
</tr>
<tr>
<td>Standard beam</td>
<td>261</td>
<td>79</td>
</tr>
<tr>
<td>Optimum beam</td>
<td>293</td>
<td>50</td>
</tr>
</tbody>
</table>

Figure 6 illustrates the failed shapes of the optimum and standard lipped channel sections at a drift ratio of 4% (i.e. the SMF limit). The von Mises stress distribution is also shown, with grey areas indicating yielding. As can be seen in Table 2, the energy dissipation capacity of the lipped channel increased by 34% after applying the shape optimization algorithm.
3. CYCLIC BEHAVIOUR OF BOLTED MOMENT CONNECTIONS USING OPTIMUM CFS CHANNELS

To evaluate the cyclic performance of connections containing the optimized beam, the bolted moment connection in Figure 1 was studied. Sufficiently thick back-to-back channels were used for the column section, so that it remained elastic during the analysis, which is in agreement with the strong-column weak-beam concept (Sabbagh et al. 2012). The FE models included material nonlinearity, initial imperfections and bearing action, all of which have previously been found to affect the accuracy of the FE model (Ye 2016).

3.1 FE modelling

The general-purpose S8R element, which is an 8-noded quadrilateral shell element with reduced integration available in the ABAQUS element library, was selected. The length and width of the elements in the mesh were 20mm. It was observed that further refining the mesh did not result in any significant increase in accuracy. The boundary conditions were applied according to the test set-up presented in Sabbagh et al. (2012). The shape of the imperfection was generated by using an eigenvalue buckling analysis in ABAQUS.

In order to model the bolt group, the point-based “Fastener” feature in ABAQUS was employed and its specifics are shown in Figure 7. For each single bolt in Figure 1, a two-layer fastener configuration was used, as shown in Figure 7. Each layer consists of a connector element connected by a fastener point to the beam section and by another fastener point to the gusset plate. The connector element allows the interaction properties of the fastener to be defined. In the definition of each fastener a “physical radius” equal to half of the bolt shank diameter was used. Each fastener point was connected to the CFS steel plates using a constraint that couples the displacements and rotations of each fastener point to the average displacements and rotations of the nodes within the physical radius.

![Diagram](image)

Figure 7. Single bolt modelling in ABAQUS: (a) definition of fastener; (b) components defined within a connector section

3.1 Hysteretic behaviour of bolted moment connections

Figure 8 and Figure 9 indicate the hysteretic performance and the failure mode, respectively, of bolted moment connections using the standard and the optimum beams. It should be noted that the bending moment due to the applied concentrated load at the end of the beam was calculated at the centre of the bolt group. Similarly to section 2.2, the bolted moment connection with the optimum beam was able to provide a 15% higher energy dissipation capacity compared to the connection with the standard beam. It should be mentioned that the local failure mode is dominant in the connection considered as the floor system was assumed to provide restraint against lateral-torsional buckling.
4. SEISMIC CHARACTERISTICS OF CFS MOMENT RESISTING FRAMES

To study the seismic characteristics of CFS moment resisting frames, a moment resisting portal frame was modelled in OpenSees using elastic beam-column elements, connected by zero length rotational spring elements which served to represent the non-linear behaviour of the structure. The springs followed a bi-linear hysteretic response, based on the Modified Ibarra Krawinkler Deterioration Model. A leaning column with gravity loads was linked to the frame by truss elements in order to simulate P- Delta effects. A schematic view of the model is presented in Figure 10. In this study, the gravity load was taken equal to 20% of compressive capacity of the column (ASCE/SEI, 41-13, 2013), which was applied on the CFS beams. The lumped plasticity non-linear model in OpenSees was used to simulate the strength and stiffness degradation in the CFS elements under earthquake excitations.
4.1 Modelling non-linear behaviour

Non-linear behaviour was modelled using the concentrated plasticity concept with rotational springs. The rotational behaviour of the plastic regions followed a bi-linear hysteretic response, based on the modified Ibarra Krawinkler deterioration model (Ibarra and Krawinkler 2005, Lignos and Krawinkler 2009 and 2010). The phenomenological IK model (see Figure 11) relies on a backbone curve that defines a boundary for the behaviour of a structural component and establishes strength and deformation bounds, as well as a set of rules that define the basic characteristics of the hysteretic behaviour between these bounds (Lignos and Krawinkler 2011).

The following regression equations were used to obtain the pre-capping plastic rotation ($\theta_p$), and the post-capping rotation ($\theta_{pc}$):

$$\theta_p = 0.0865 \left( \frac{h}{t_w} \right)^{-0.365} \left( \frac{b}{2t_f} \right)^{-0.14} \left( \frac{L}{d} \right)^{0.34} \left( \frac{c_{unit}^1 d}{533} \right)^{-0.721} \left( \frac{c_{unit}^2 F_{y}}{355} \right)^{-0.23}$$

$$\theta_{pc} = 5.63 \left( \frac{h}{t_w} \right)^{-0.565} \left( \frac{b}{2t_f} \right)^{-0.8} \left( \frac{c_{unit}^1 d}{533} \right)^{-0.28} \left( \frac{c_{unit}^2 F_{y}}{355} \right)^{-0.43}$$

The rate of cyclic deterioration was defined by the reference cumulative plastic rotation ($\Lambda$)
parameter determined as follows:

\[ \Lambda = \frac{E_t}{M_y} = 495 \left( \frac{h}{t_w} \right)^{1.34} \left( \frac{b_f}{2t_f} \right)^{-0.595} \left( \frac{c_{\text{unit}}^2 F_y}{355} \right)^{-0.36} \]  

(5)

In the above equations, \( t_w \) is the thickness of the web, \( h \) is the depth of the web, \( t_f \) is the thickness of the flange, \( b_f \) is the width of the flange, \( d \) is the depth of the section, \( F_y \) is the yield strength of the section, \( L \) is the distance from the plastic hinge to the point of inflection and \( L_0 \) is the distance from the connection face to the nearest lateral brace. Moreover, \( c_{\text{unit}}^1 \) and \( c_{\text{unit}}^2 \) are coefficients for unit conversion. They are both 1.0 if millimetres and megapascals are used, and \( c_{\text{unit}}^1 = 25.4 \) and \( c_{\text{unit}}^2 t = 6.895 \) if \( d \) is in inches and \( F_y \) is in ksi. In addition, it is assumed that \( \frac{M_x}{M_y} = 1.10 \), as suggested by Lignos and Krawinkler (2010).

It should be mentioned that while in general CFS elements may exhibit buckling before formation of full plastic hinges, previous studies showed that a good plastic behaviour will be observed if the elements are thick enough and designed properly (Mojtabaei et al. 2017).

### 4.2 Ductility and energy dissipation of CFS moment-resisting frames

Ductility is defined as the ability of a structure to undergo plastic deformations without significant loss of strength, which is especially important in controlling the structural damage during strong earthquakes. The ductility ratio of CFS moment resisting frames can be determined using the following equation:

\[ \mu = \frac{\Delta_u}{\Delta_y} \]  

(6)

Where \( \Delta_y \) and \( \Delta_u \) are the yield and the ultimate displacement of the frame, respectively. The ultimate displacement was assumed to correspond to the displacement at which the load carrying capacity of the system has dropped by 20% relative to the peak load. In this study, the yield displacement was determined based on the ASCE/SEI 41-13 (and FEMA 356) recommended method, in which the load-displacement curve is represented by a bilinear curve with a post-yield slope as shown in Figure 12. It should be mentioned that the area under the hysteretic curve is generally identified as the energy dissipation capacity of the structures.

![Lateral seismic force](image)

(a) Positive post-yield slope  
(b) Negative post-yield slope

Calculation of ductility based on FEMA’s concept

Figure 12. Definition of the yield and ultimate displacements for calculation of ductility.

Table 3 lists the energy dissipation capacity and the ductility of the studied CFS moment resisting frame using the standard and the optimum CFS beams. It can be seen that the frame with optimized beams provided a 29% higher energy dissipation capacity compared to the standard CFS frame. However, the
ductility of the optimum frame dropped 13% compared to the standard frame. It should be noted that the CFS beam elements in this study were optimised to have maximum energy dissipation capacity over the cyclic loading history, and therefore, the ductility of the whole structural system may be slightly reduced. However, if the ductility demand under the design earthquake does not exceed the target ductility, the optimum design frame will exhibit considerably lower structural damage compared to its conventionally designed counterpart by dissipating higher energy levels.

Table 3. Energy dissipation and ductility of moment resisting portal frames with the standard and optimum beams

<table>
<thead>
<tr>
<th>Beam type</th>
<th>Energy dissipation (kJ)</th>
<th>Ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard beam</td>
<td>26132</td>
<td>4.3</td>
</tr>
<tr>
<td>Optimum beam</td>
<td>31241</td>
<td>3.9</td>
</tr>
</tbody>
</table>

5. CONCLUSIONS

This paper presents a study of the energy dissipation capacity and ductility of CFS channel sections, used as part of Moment Resisting Frames in seismic applications. An optimisation framework is presented, which integrates a PSO algorithm with detailed GMNIA finite element cyclic analyses in order to produce CFS lipped channels with optimum energy dissipation characteristics. Practical construction and manufacturing constraints were also considered. The cyclic performance of bolted moment connections was assessed for the standard and optimised beams using an experimentally validated model. Finally, the non-linear behaviour at the frame level was modelled utilizing the concentrated plasticity concept with rotational springs based on the modified Ibarra and Krawinkler model (2005). Based on the analysis results it was concluded that, at the element level, the energy dissipation capacity of the CFS lipped channel increased by 34% using shape optimisation, while using bolted-moment connection with optimum beam leads to 15% growth in energy dissipation compared to the connection with standard beam. At the frame level, the optimum frame provided 29% more energy dissipation than the standard CFS frame. However, by focusing the optimisation on the energy dissipation capacity, the ductility of the optimum frame dropped by 13% compared to the standard frame.

6. REFERENCES


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