Fire design proposal for cold-formed steel lipped channel beam-columns

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ABSTRACT: Steel structural elements composed of cold-formed thin-walled sections are commonly applied in buildings due to their efficiency in terms of stiffness and strength. However, under fire conditions the thin walls of these profiles, along with the steel high thermal conductivity, induce fast increase on the steel temperature and corresponded significant loss of those referred stiffness and strength. This paper presents a numerical study on the members behaviour with cold-formed lipped channel sections at high temperatures, when subjected to bending plus compression. Comparisons between the finite element results and the analytical methodologies from EN1993-1-2 are presented, using its Annex E as well as its French National Annex, where different reduction factors for the steel constitutive law and different design formulae are recommended for cold-formed profiles. Both methodologies revealed to be safe but sometimes too conservative, which motivated the development of a new design proposal.

1 INTRODUCTION

Cold-formed steel profiles application on structures has been gaining popularity in construction industry due to their lightness, high strength-weight ratio, faster manufacturing process, easy prefabrication and mass production, contributing positively to the sustainability of buildings. These elements are typically used in steel structures as joists, tracks, studs or girts, subjected to different loading conditions (Schafer, 2011, Dubina et al., 2012).

Open cold-formed steel members have high susceptibility to the occurrence of different instability phenomena, such as local, global and also distortional buckling. The influence of these instability phenomena on the ultimate bearing capacity of beam-columns, composed of those cold-formed sections, at normal temperature have been widely studied (Torabian et al., 2015, Li et al., 2016). However, and although it is known that elevated temperatures impose significant loss of strength and stiffness on cold formed members (Kankanamge & Mahendran, 2011, Ranawaka & Mahendran, 2009), the behaviour under fire situation of these structural elements, subjected to bending plus compression, is still less known.

Hence, this paper presents a numerical parametric study on the behaviour of laterally restrained double hinged members with cold-formed lipped channel sections in case of fire, subjected to bending plus compression, considering different member and section slendernesses, bending moment diagrams, steel grades and elevated temperatures.

The members ultimate bearing capacity is obtained through numerical modelling, considering geometrically and materially nonlinear analyses including imperfections (GMNIA), using the Finite Element software SAFIR (Gernay & Franssen, 2017). The different instability modes (local, distortional and global buckling), needed for the definition of initial geometric imperfections, are obtained using the software CAST3M (CAST3M, 2012) and applying RUBY interface (Couto et al., 2013).

Comparisons between the finite element results and the analytical methodologies from EN1993-1-2 (CEN, 2005) rules are here presented, using its Annex E for Class 4 cross-sections and its French National Annex (CEN, 2007), where different reduction factors for the steel constitutive law and different design formulae are recommended for cold-formed profiles. Following the obtained observations, a new proposal for the safety evaluation of these cold-formed beam-columns at elevated temperatures is presented and analysed here.

2 FIRE DESIGN RULES

The EN1993-1-2 recommends the use of the 0,2% proof strength at temperature θ instead of the strength at 2% total strain at temperature θ , applied to non-slender sections (Class 1, 2 or 3), for members with Class 4 cross-sections (common on cold-formed steel profiles).

The expression for laterally restrained steel beam-columns subjected to bending about the major axis in fire situation considered here is:

$$\frac{N_{fi,Ed}}{\chi_{y,fi} A_{eff} k_{y,\theta} \frac{f_y}{\gamma_{M,fi}}} + k_y \frac{M_{y,fi,Ed}}{W_{eff,y} k_{y,\theta} \frac{f_y}{\gamma_{M,fi}}} + k_z \frac{\Delta M_{z,fi,Ed}}{W_{eff,z} k_{y,\theta} \frac{f_y}{\gamma_{M,fi}}} \leq 1.0 \quad (1)$$

with A_{eff} (effective area) and $W_{eff,y}$ (effective section modulus) the cross-section effective properties, $\Delta M_{z,fi,Ed}$ the additional bending moment due to the shift of the centroidal axis, and where the reduction factor χ_{fi} value is obtained according to equation (2):

$$\chi_{fi} = \frac{1}{\phi_\theta + \sqrt{[\phi_\theta]^2 - [\bar{\lambda}_\theta]^2}} \leq 1.0 \quad (2)$$

with:

$$\phi_\theta = \frac{1}{2} \left[1 + \alpha \bar{\lambda}_\theta + (\bar{\lambda}_\theta)^2 \right] \quad (3)$$

and α an imperfection factor determined by:

$$\alpha = 0.65 \sqrt{\frac{235}{f_y}} \quad (4)$$

The recommended interaction factor, for Class 4 cross-sections, is obtained from:

$$k_i = 1 - \frac{\mu_i N_{fi,Ed}}{\chi_{i,fi} A_{eff} k_{0.2p,\theta} f_y / \gamma_{M,fi}} \leq 3 \text{ with } i = y \text{ or } z \quad (5)$$

being μ (for the strong axis) given by:

$$\mu_y = (2\beta_{M,y} - 5) \bar{\lambda}_{y,\theta} + 0.44\beta_{M,y} + 0.29 \leq 0.8 \text{ with } \bar{\lambda}_{y,20^\circ\text{C}} \leq 1.1 \quad (6)$$

$$\mu_z = (1.2\beta_{M,z} - 3) \bar{\lambda}_{z,\theta} + 0.71\beta_{M,z} - 0.29 \leq 0.8 \quad (7)$$

and $\beta_{M,i}$, which is in function of the bending diagram shape, equal to:

$$\beta_{M,i} = 1.8 - 0.7\psi \quad (8)$$

where ψ is the ratio between the end moments ($\psi = M_1/M_2$).

The French National Annex (FN Annex) of EN1993-1-2 proposes a similar calculation methodology, considering for cold-formed steel profiles $k_i = 1$ and lower reduction factors for steel yield strength and young modulus at high temperatures, when compared to Annex E of EN1993-1-2 (Zhao et al., 2005). Moreover, for flexural buckling the following equation should be used:

$$\phi_\theta = \frac{1}{2} \left[1 + \alpha(\bar{\lambda}_\theta - 0.2) + (\bar{\lambda}_\theta)^2 \right] \quad (9)$$

considering the imperfection factor α obtained from EN1993-1-3 (CEN, 2006a), where curve b ($\alpha = 0.34$) is proposed for lipped channel sections, and $\bar{\lambda}_\theta$ the relative slenderness for flexural buckling at the steel temperature θ . A new proposal is presented here based in an adaptation methodology for flexural buckling on cold-formed steel lipped channel cross-section elements, proposed in previous studies (Arrais, 2018), where the equations considered in this new proposal remain the same from EN1993-1-2 considering the use of interaction equation (1) with the flexural buckling curve of FN Annex (equation 9) and a new limit for equation (6) of $\mu_y = 1.1$.

3 CASE STUDY AND NUMERICAL MODELS

The cold-formed lipped channel sections (C) presented on Table 1 were chosen from different references for the numerical analysis of beam-columns without lateral torsional buckling (LTB). Parameters such as the steel grade (S280, S320, S355 and S460), member slenderness (0.5 m, 1.5 m, 3.0 m, 4.0 m, 5.0 m and 10.0 m) and bending diagrams (uniform bending, $\psi = 1$, and non-uniform bending $\psi = 0$ and $\psi = -1$) are studied here. The temperatures of 350 °C, 500 °C and 600 °C were applied and considered uniform throughout the cross section.

In the finite element model (Figure 1), rectangular shell finite elements with four nodes, with six degrees of freedom (three translations and three rotations), of 10 mm length over the member cross-section at flat plates, are used. The mesh was refined at the corners in order to better reproduce the rounded shape. In the longitudinal direction, the mesh size varies between 10 mm (for small lengths) and 25 mm (for the longest member). For imposing end moments, loads in the parallel direction to the beam-column axis were applied on the nodes at the element ends, according to the linear stresses distribution resulting from simple bending around the strong axis. Restrictions are applied to reproduce simple supports on beam-column ends.

This study did not take into account the corner enhancement, resulting from the manufacturing process, since this increase is not considered in EC3 (CEN, 2006a) in the occurrence of instability phenomena and because its influence is reduced at high temperatures (Arrais et al. 2019). Residual stresses were considered according to Schafer & Peköz (1998).

Table 1. Lipped channel sections analysed and respective dimensions.

	Web	Flange	Lip	Thickness
Designation	mm	mm	mm	mm
C_229x64x20x(1.5)*	229	64	20	1.5
C_154x35x11x(1.5)**	154	35	11	1.5
C_150x43x20x(2.0)***	150	43	20	2.0

* (Schafer & Peköz, 1998), ** (Torabian et al. 2015), *** (PERFISA, 2014)

Figure 1. Numerical model adopted for analysed beam-columns.

Table 2. Geometric imperfections' magnitudes considered.

Local	Distortional	Global
$0.80 \frac{b}{100}$	$0.80 \frac{b}{100}$	$0.80 \frac{L}{750}$

The shapes obtained from local, distortion and global buckling modes, applying CAST3M program together with the RUBY interface, were used to define the initial geometric imperfections. According to Annex C of EN 1993-1-5 (CEN, 2006b), the geometric imperfections' magnitudes were obtained from 80% of the geometric manufacturing tolerances prescribed in EN 1090-4 (CEN, 2018). Table 2 shows the maximum amplitude of these imperfections where b is the length of the web or flange, depending on the highest deformation value, and L is the member length.

A geometric imperfection's combination is introduced in the numerical model. EN 1993-1-5 states that a leading imperfection should be chosen, and the accompanying imperfections may have their value reduced to 70%. In this work, the leading imperfection was chosen according to the achieved lower resistances from each individual imperfection obtained resistance.

The applied steel material laws follow EN 1993-1-2 steel constitutive law model. However, and as aforementioned, FN Annex proposes for cold-formed profiles the use of lower values for the reduction factors of the yield strength and elasticity modulus at elevated temperatures. The comparison between the EN 1993-1-2 and FN Annex constitutive laws is presented in Figure 2.

Figure 2. Constitutive law for elevated temperatures according to EN1993-1-2 and respective FN Annex.

4 COMPARISON BETWEEN NUMERICAL RESULTS AND FIRE DESIGN RULES

The numerically obtained ultimate bearing load capacities are here compared with the calculation rules of EN 1993-1-2. Since the analysed profile is composed of a monosymmetric section, the change of the centre of gravity, due to the consideration of the effective area A_{eff} , results in an eccentricity of the applied axial force, which induces an additional, $\Delta M_{z,fi,Ed}$, acting bending moment about the weak axis. For illustrating the referred comparisons for a C229x64x20 with 1.5 m length at 500 °C with different load levels, this additional moment was added to the axial force buckling resistant capacity on the vertical axis of the charts in Figures 3 (uniform bending) and 4 (triangular bending diagram). Figure 5 illustrates the failure mode of tested beam-columns.

Figure 3. Comparison between the numerical results and the design rules of the EN1993-1-2 Annex E, the French National Annex (FNA) and the New Proposal, for uniform bending ($\psi = 1$), at 500 °C.

Figure 4. Comparison between the numerical results and the design rules of the EN1993-1-2 Annex E, the French National Annex (FNA) and the New Proposal, for non-uniform bending ($\psi = 0$), at 500 °C.

Figure 5. Collapse mode of a C_229x64x20 section beam-column model.

Table 3. Statistical evaluation considering uniform bending diagram on a total of 1176 results.

Design rule	Average value (μ)	Standard deviation (s)	Máx. unsafe	% unsafe
EN1993-1-2 (EN1993-1-2 Constitutive Law)	0.93	0.10	-	0.00
EN1993-1-2 FNA (FN Annex Constitutive Law)	1.09	0.16	1.05	18.23
New Proposal (FN Annex Constitutive Law)	0.97	0.03	1.02	6.74

Table 4. Statistical evaluation considering non-uniform bending diagram results on a total of 1512 results.

Design rule	Average value (μ)	Standard deviation (s)	Máx. unsafe	% unsafe
EN1993-1-2 (EN1993-1-2 Constitutive Law)	0.67	0.30	-	0.00
EN1993-1-2 FNA (FN Annex Constitutive Law)	1.31	0.33	1.07	12.37
New Proposal (FN Annex Constitutive Law)	0.81	0.28	1.01	6.51

Following the criteria of the validation proposed by Kruppa (1999), Table 3 presents the statistical evaluation for uniform bending moments based on a total of 1176 results (considering both constitutive laws) and Table 4 for non-uniform bending moments based on a total of 1512 results.

From the presented comparisons, it can be concluded that both approaches are mostly on the safety side, being however too conservative. Considering FN Annex, the values are closer to the EC3 curve for $\psi = -1$, whereas in the case of non-uniform moments, the curve remains very conservative, as in this methodology the interaction factor k_y and k_z are always one. The proposed adjustments are more accurate and safe when compared to the numerical results.

5 CONCLUSIONS

In the present work, a parametric study of the fire behaviour of cold-formed beam-columns with lipped channel (C) sections, without lateral torsional buckling, was presented, considering the EN1993-1-2 as well as its French National Annex constitutive laws.

A comparison was made between the ultimate load capacities obtained numerically and the requirements of EC3, concluding that these last calculation rules are on the safety side in most cases, being too conservative. The comparison with the FN Annex demonstrated that the values are closer to the FNA curve, whereas in the case of non-uniform moments, the curve remains very conservative. The results were obtained considering the variation of several parameters such as different steel grades, bending moment diagrams and slendernesses.

Thus, a new design proposal was developed with some adjustments in relation to the current calculation rules. It was possible to observe that the new proposal presents a better approximation to the numerical results when compared to EC3.

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