Parametric Study on the Interaction between Axial Compression and Bending on Austenitic Stainless Steel Members with Hollow Sections under Fire Situation

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Abstract

This research work presents a parametric numerical study on the resistance at elevated temperatures of stainless steel members, subjected to combined bending and axial compression. Previous studies have shown the need for the development of further studies aiming at better predicting the fire behaviour of stainless steel beam-columns. However they have only considered beam-columns composed of stocky I sections. Hence, this paper focuses on austenitic stainless steel (European grade 1.4301 also known as 304) beam-columns composed of square hollow sections (SHS) and circular hollow sections (CHS), considering different cross-sections classes (1 to 4), according to Eurocode 3 (EC3) classification.

The numerical analyses were performed using the finite element program SAFIR, with material and geometric non-linear analysis considering imperfections. The influence of the following parameters was evaluated: bending moment diagram shape, cross-section slenderness considering the local buckling occurrence on the thin-walled sections, and member slenderness for the global instability due to flexural buckling.

Comparisons between the obtained numerical results and the interaction curves of Eurocode 3 are presented. The results show that specific design approach should be developed for these stainless steel members under fire situation, taking into account the above mentioned parameters.

Keywords

fire resistance; stainless steel; beam-column; buckling; numerical analysis.

1 Introduction

There has been an increase in the use of stainless steel in construction for structural purposes [1].The high number of desirable characteristics that stainless steel possesses for its structural use, such as durable character, aesthetic excellence and easy maintenance, are some of the main reasons for this increasing use. Although the initial investment is higher when compared to carbon steel application, the stainless steel can be competitive, due to its low life cycle cost, contributing to more sustainable construction [2]. In addition, stainless steel has also a higher fire resistance than the one of carbon steel [3]. However, it is still necessary to develop knowledge on its structural behaviour at elevated temperatures, as existing fire design guidelines, such as in Part 1-2 of EC3 [3], are still based on the formulations developed for carbon steel members in spite of their different material behaviour.

The beam-column structural element is the most common in construction. These members are elements subject to combined bending and axial compression. Bending can be applied by different bending moment diagrams shape and due to the compression stresses beam-columns are susceptible to buckling phenomena occurrence.

The shape and walls slenderness of the cross-sections play also important roles on the member behaviour. Cold form hollow sections are also often chosen for stainless steel columns. Normally, these sections exhibit specific characteristics that directly affect the final resistance of the corresponded members, such as the corners strength enhancement due to the fabrication process in quadrangular and rectangular section shapes [4]. In EC3 [5,6] the walls slenderness determines the cross-section classification (from Class 1 to Class 4). Elements composed of Class 4 cross sections are more susceptible to the occurrence of local buckling failure mode in addition to the flexural buckling, when compared to the other cross sections classes, due to the thin walls that are associated to this type of cross section.

The behaviour of stainless steel columns in case of fire has recently been given more attention. Several studies have been performed in stainless steel columns resulting in different new proposals. Some of them were based on columns with Class 1 and 2 (stocky) I cross-sections [7] (as the EC3 [3] fire design rules for carbon steel), and others on columns with hollow sections in case of fire [8,9,10,11]. However, interaction behaviour between axial compression and bending in beam-columns at elevated temperatures has still not been completely understood. Some studies on Class 1 and 2 (stocky) I cross-sections [12] and preliminary analysis to members with Class 4 square hollow sections [13] have been performed, concluding that further parametric studies should be developed.

The main objective of this work is to present a parametric study based on numerical analyses on stainless steel structural elements, with square and circular hollow cross-sections, subjected to axial compression plus bending under fire conditions, applying geometrically and materially non-linear imperfect analysis with the program SAFIR [15]. The accuracy and safety of EC3 interactions curves for structural elements subjected to bending plus axial compression in case of fire is here analysed. An extensive campaign of parametric numerical simulations has been performed on Class 1, 2, 3 and 4 stainless steel square and circular hollow sections at elevated temperatures, considering three bending moment diagram shapes corresponded to end moments (uniform, triangular and bi-triangular shapes). In this study the stainless steel grade 1.4301 (most used stainless steel grade for structural applications) was considered. Comparisons, between the numerical results and the EC3 rules (EN 1993-1-2 [3]) and the interaction curves recommended in Part 1-1 of EC3 [5] and in Part 1-4 of EC3 [6], are made.

2 Simplified Calculation Methods for Fire Design

In this section the considered methodologies in this study, for the calculation of stainless steel beam-columns resistance under fire situation, are presented.

For the cross-section classification, the following equation was used to determine the factor ε, according to [14].

$$\varepsilon = 0.85\left[\frac{235}{f_y}\frac{E}{210000}\right]^{0.5} \tag{1}$$

Where f_y and E are respectively the yield strength and Young modulus in MPa.

2.1 Cross-section resistance

According to EN 1993-1-2 [3], the section resistance of a stainless steel member is calculated in the same way as for carbon steel, changing only the mechanical properties of the material, such as yield strength and modulus of elasticity to consider uniform elevated temperatures in the section, resulting from a fire.

The expressions used in the interaction curves when the section is subjected to bending plus axial compression for square hollow sections and circular hollow sections are presented in Table 1.

Table 1 Formulations for cross-section resistance of structural hollow sections

	Square hollow sections	Circular hollow sections
Class 1 and 2 sections	$\frac{M_{y,Ed}}{M_{N,y,Rd}} \le 1$ with $M_{N,y,Rd} = \frac{M_{pl,y,Rd}\left(1-\frac{N_{Ed}}{N_{pl,Rd}}\right)}{\left(1-0.5min\left(\frac{A-2bt}{A};0.5\right)\right)} \le M_{pl,y,Rd}$	$\frac{M_{y,Ed}}{M_{N,y,Rd}} \le 1$ with $M_{N,y,Rd} = M_{pl,y,Rd}\left(1-\left[\frac{N_{Ed}}{N_{pl,Rd}}\right]^{1.7}\right) \le M_{pl,y,Rd}$
Class 3 and 4 sections	$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} \le 1$ Where: N_{Rd} is determined using the gross cross section area (A) for Class 3 sections and the effective section area (A_{eff}) for Class 4 sections; $M_{y,Rd}$ is determined using the elastic section modulus ($W_{el,y}$) for Class 3 sections and the effective section modulus ($W_{eff,y}$) for Class 4 sections.	

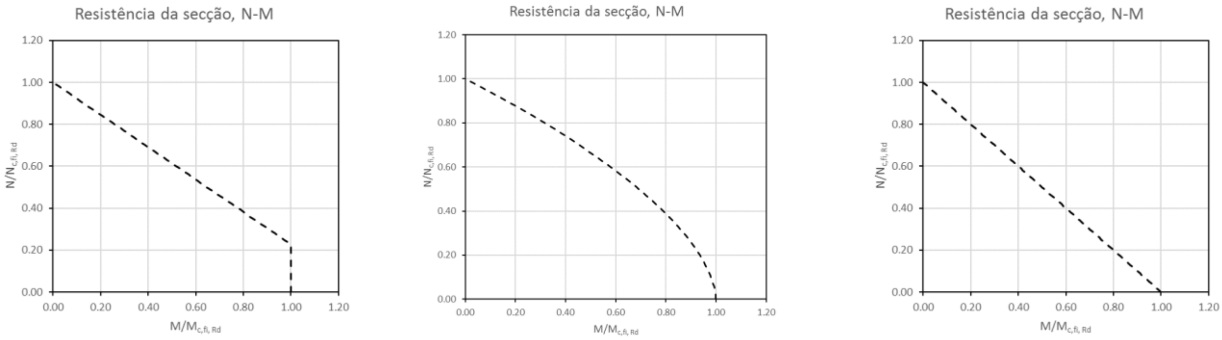
In a fire situation higher strains than at room temperature are acceptable, therefore, instead of 0.2% proof strength usually considered at normal temperature, for cross-section of classes 1, 2 and 3 at elevated temperatures the stress at 2% of total strain should be considered as the yield strength (Eq. 2).

$$f_{y,\theta} = f_{2\%,\theta} = k_{2\%,\theta} f_y \tag{2}$$

However, for Class 4 cross-sections, according to Annex E of EN 1993-1-2, the proof strength at 0.2% plastic strain should be applied (Eq. 3).

$$f_{y,\theta} = f_{0.2p,\theta} = k_{0.2p,\theta} f_y \tag{3}$$

The resulting interaction curves are shown in Fig. 1.



a) b) c)

Fig. 1 Interaction curves for: a) Class 1 and 2 cross-sections of SHS; b) Class 1 and 2 cross-sections of CHS; c) Class 3 and 4 cross-sections of both SHS and CHS.

2.2 Interaction curves for member resistance

The EC3 interaction formulae for beam columns in case of fire [3] were developed based on carbon steel columns with I sections of Classes 1 and 2 [17]. Due to the different material behaviour of stainless steel, previous research works [12] have proposed new interaction curves for stainless steel members based on parametric studies also on stainless steel I sections of Classes 1 and 2. Additionally, Part 1-1 of EC3 [5] and Part 1-4 of EC3 [6] proposes different methods for evaluating the resistance of carbon steel and stainless steel beam columns at normal temperature.

In this section the expressions for the determination of those different interaction curves are presented. The same main design expression (4) is used in all cases, changing only the interaction factor k. As the analysed sections are square and circular, the proposed formulae for the strong axis on the design recommendations were used (k_y or k_{yy} on the case of Part 1-1 of EC3).

$$\frac{N_{fi,Ed}}{\chi_{fi} A \dfrac{f_{y,\theta}}{\gamma_{M,fi}}} + k \frac{M_{fi,Ed}}{W \dfrac{f_{y,\theta}}{\gamma_{M,fi}}} \leq 1 \tag{4}$$

As in the cross-section resistance expressions, plastic geometric properties should be considered for Class 1 and 2 sections, elastic geometric properties for Class 3 sections and effective geometric properties plus the proof strength at 0.2% for Class 4 sections.

The reduction factor for flexural buckling under fire design situation χ_{fi} is given by

$$\chi_{fi} = \frac{1}{\phi_\theta + \sqrt{(\phi_\theta)^2 - (\bar{\lambda}_\theta)^2}} \quad \text{with } \chi_{fi} \leq 1 \tag{5}$$

Being

$$\phi_\theta = \frac{1}{2}\left[1 + \alpha\bar{\lambda}_\theta + (\bar{\lambda}_\theta)^2\right] \tag{6}$$

where the imperfection factor α depends on the steel grade and is determined according to:

$$\alpha = 0.65\sqrt{\frac{235}{f_y}} \tag{7}$$

The non-dimensional slenderness $\bar{\lambda}_\theta$ for high temperatures is given by:

$$\bar{\lambda}_\theta = \bar{\lambda}\sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}} \tag{8}$$

where $\bar{\lambda}$ is the non-dimensional slenderness at normal temperature and $k_{y,\theta}$ and $k_{E,\theta}$ the reduction factors for the yield strength (according to equation 2 or 3) and for the Young modulus.

On the study presented in this paper, for better comparisons purposes, the adopted flexural buckling reduction factor was obtained numerically from the resistance of axially compressed columns.

In order to analyse the accuracy and safety of different design approaches, different interaction factor k corresponded to the following methodologies were considered.

- EN 1993-1-2 [3];

- EN 1993-1-4 [6] adapted at elevated temperatures;
- EN 1993-1-1 [5] adapted to stainless steel at elevated temperatures;

EN 1993-1-2

Part 1-2 of EC3 [3], in its Annex C, states that the safety evaluation of stainless steel members at elevated temperatures should be made following the same expressions developed for fire design of carbon steel members. The recommended interaction factor is

$$k = 1 - \frac{\mu N_{fi,Ed}}{\chi_{fi} A \frac{f_{y,\theta}}{\gamma_{M,fi}}} \leq 3 \tag{9}$$

Where

$$\mu = (2\beta_M - 5)\bar{\lambda}_\theta + 0.44\beta_M + 0.29 \leq 0.8 \ \text{ with } \ \bar{\lambda}_{20ºC} \leq 1.1 \tag{10}$$

β_M, which is in function of the bending diagram shape, is for end moments given by

$$\beta_M = 1.8 - 0.7\psi \tag{11}$$

Being ψ the ratio between the end moments.

EN 1993-1-4 adapted at elevated temperatures

Part 1-4 of EC3 [6] is dedicated to the structural design of stainless steel structures at normal temperature. The proposed interaction factor for members subjected to combined bending plus axial compression is

$$k = 1 + 2(\bar{\lambda} - 0.5)\frac{N_{Ed}}{N_{b,Rd}} \quad \text{but} \quad 1.2 \leq k \leq 1.2 + 2\frac{N_{Ed}}{N_{b,Rd}} \tag{12}$$

As it can be observed, this expression does not depend on the bending diagram shape, which is normally an important parameter for the calculation of beam-columns resistance.

The proposed minimum limit to the interaction factor of 1.2 was not used in this study as it prevents the attainment of the total bending resistance, when no axial compression is applied.

The Adaptation to fire of these expressions is made through the application of the yield strength (equations 2 and 3) and Young modulus at elevated temperatures, proposed in Annex C of Part 1-2 of EC3 [3].

EN 1993-1-1 adapted to stainless steel at elevated temperatures

The Part 1-1 of EC3 [5] presents two methods for the determination of the interaction factors on carbon steel beam-columns. In this work it is evaluated the possibility of using the expressions corresponded to one of these methods (Method 2) on stainless steel members with hollow sections submitted to fire.

According to the Annex A of EN 1993-1-1 the interaction factor can be determined for Class 1 and 2 sections by:

$$k = c_m\left(1 + (\bar{\lambda} - 0.2)\frac{N_{Ed}}{N_{b,Rd}}\right) \leq c_m\left(1 + 0.8\frac{N_{Ed}}{N_{b,Rd}}\right) \tag{13}$$

And for Class 3 and 4 sections by

$$k = c_m\left(1 + 0.6\bar{\lambda}\frac{N_{Ed}}{N_{b,Rd}}\right) \leq c_m\left(1 + 0.6\frac{N_{Ed}}{N_{b,Rd}}\right) \tag{14}$$

Being for, end moments, c_m given by

$$c_m = 0.6 + 0.4\psi \geq 0.4 \tag{15}$$

In this work, the yield strength and Young modulus values applied on these formulae were the one proposed in Part 1-4 of EC3 [6] being reduced at elevated temperatures as proposed in Annex C of Part 1-2 of EC3 [3].

3 Numerical Modelling

In this section the chosen study cases and the adopted numerical model used on the parametric study are described.

3.1 Case study

In this work doubled hinged columns, of the austenitic stainless steel grade 1.4301 (also known as 304), subjected to compression plus bending diagrams were analysed.

The three shapes of bending diagrams shown in Fig. 2 were considered: uniform bending (ψ=1), triangular diagram (ψ=0) and bi-triangular diagram (ψ=-1).

Four square hollow sections and four circular hollow sections were chosen in this study in order to cover the different section classifications. Two sections of Class 4 were chosen, one less slender and another more slender. The same width for the SHS (200 mm) and the same diameter for the CHS (244.5 mm) were maintained being change the thicknesses for obtaining the different sections (Table 2).

Due to the thin walls of these profiles and to the stainless steel high thermal conductivity, the numerical tests were made with uniform temperatures in the cross section. The temperatures chosen were: i) 350 ºC, which is the critical temperature suggested in EC3 [3] for Class 4 profiles when no calculation is performed; ii) 500 ºC for being a typical critical temperature in steel structural elements subjected to instability phenomena; and iii) 600 ºC for covering the range of common critical temperatures in steel structural elements.

For each of the above mentioned cases, 4 columns lengths were chosen (1, 3, 7 and 11 metres), corresponding to slenderness values at high temperatures (as defined in EC3 [3]) lower than 2.0.

Table 2 Tested cross-sections

Section classification	SHS [mm]	CHS [mm]
Class 1 or Class 2 sections	200x200x10	244.5x8
Class 3 sections	200x200x7	244.5x2
Class 4 sections less slender	200x200x4	244.5x1.5
Class 4 sections more slender	200x200x2	244.5x1

3.2 Numerical model

The numerical model was developed to be applied on the finite element software SAFIR [15]. Numerical analyses applying this code were recently validated against experimental fire tests in stainless steel columns [11]. This section presents the adopted material model, mesh and restrictions, geometric initial imperfections, residual stresses and corner strength enhancement (for the square hollow sections) considered in this research work.

Material model

The numerical modelling of stainless steel material law at elevated temperatures in SAFIR was made by a non-elastic plane stress condition, based on the von Mises surface and isotropic hardening [18].

Fig. 2 illustrates the applied stainless steel stress-strain relationship at elevated temperatures [3], according to Part 1-2 of EC3 prescriptions, which are characterised by having always a non-linear behavior with an extensive hardening phase, when compared with carbon steel material law.

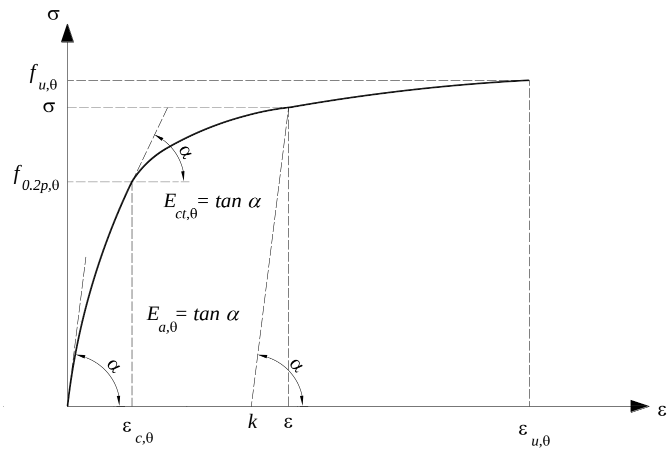


Fig. 2 Stress-strain relationship of the stainless steel at high temperatures.

Table 3 presents the reduction factors at the elevated temperatures [3], for the different parameters necessary to determine the stress-strain relationship illustrated in Fig. 2. Only the values for the three temperatures applied in this study are shown.

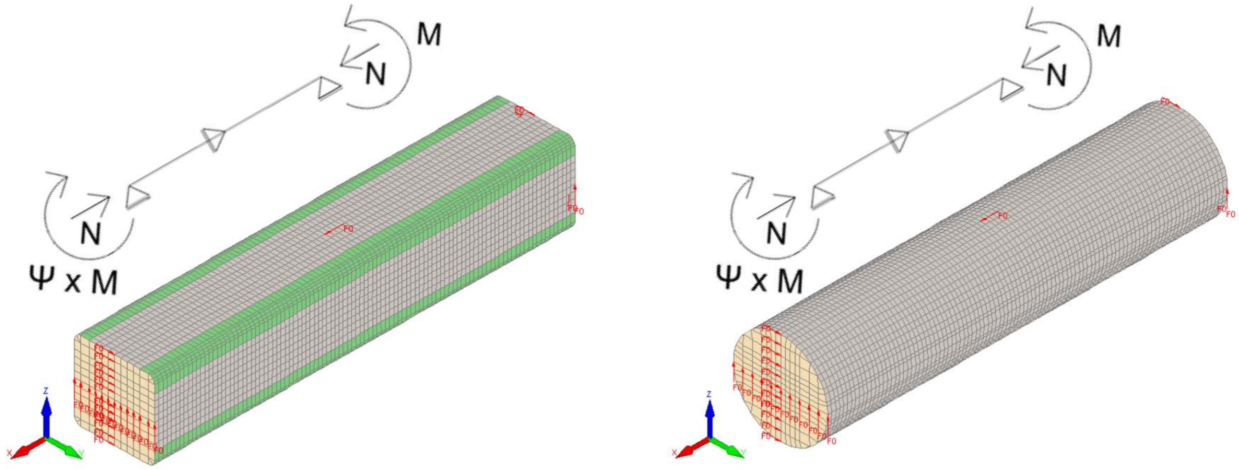
Table 3 Reduction factors for the austenitic grade 1.4301 at high temperatures

θ_a	$k_{E.\theta} = E_{a.\theta} / E_a$	$k_{0.2p.\theta} = f_{0.2p.\theta} / f_y$	$k_{u.\theta} = f_{u.\theta} / f_u$	$k_{Ect.\theta} = E_{ct.\theta} / E_a$	$\varepsilon_{u.\theta}$
350 °C	0.86	0.62	0.725	0.02	0.4
500 °C	0.8	0.54	0.67	0.02	0.4
600 °C	0.76	0.49	0.58	0.02	0.35

Mesh and restrictions

As local buckling phenomena were likely to occur on the most slender cross-sections, shell finite elements were used. The mesh size was chosen in order to capture all the possible failure modes.

The restrictions and loads applied are schematised in Fig. 3. The restriction to the longitudinal movement was imposed at the mid length, being the lateral restrictions applied on the end extremities. For the application of the loads thick end plates at 20 ºC were used for avoiding local instabilities on the mesh.



a) b)

Fig. 3 Restrictions and load applications on the model: a) Square hollow profile; b) Circular hollow profile.

The calculation of the cross-section resistances were numerically made, by imposing lateral restrictions in all member length on the corners of the square hollow sections and on four lines, parallel to the longitudinal axis, on the circular hollow sections.

Geometric initial imperfections

The adopted initial imperfections follow the recommendations of Annex C of Part 1-5 of EC3 [16], which proposes the use of the shape of the buckling modes with the amplitude equal to 80% of the manufacturing geometry tolerances that can be found in EN 1090-2:2008+A1 [19] and EN10219-2 [20]. Two instabilities modes are expected, the flexural global buckling and the local buckling.

Therefore for the global buckling the maximum amplitude adopted was 80% of $\Delta = \frac{L}{750}$ for both the squares and circular hollow profiles. For the local buckling the maximum amplitude adopted on the square hollow profiles was 80% of $\Delta = 0.008b$ and on the circular hollow profiles was 80% of $\Delta = 0.008d$. Figures 4 and 5 show on an amplified scale the used imperfections.

According to Part 1-5 of EC3 a combination of the previous described imperfections should be introduced in the model. This combination should have a main imperfection which is added to the remaining imperfection reduced to 70%. The consideration of this combination produced very small differences when compared to simply adding the two imperfections. Therefore, this last procedure was adopted.

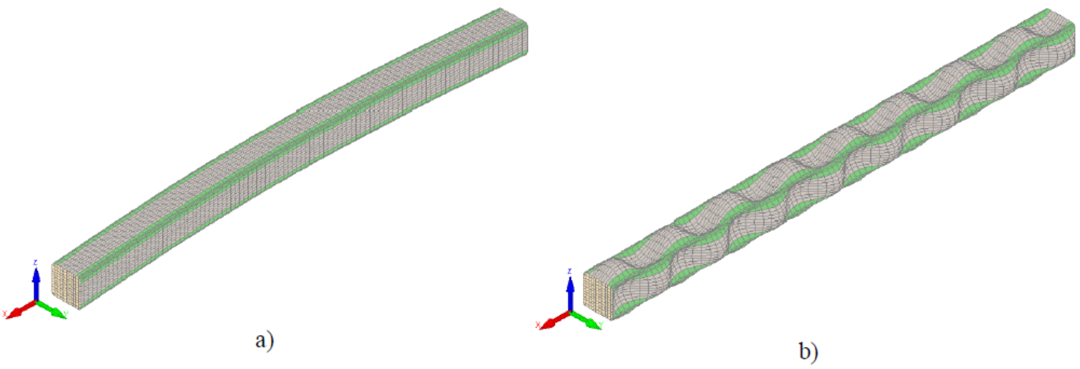


Fig. 4 Initial geometric imperfections on the square hollow profiles: a) associated to global buckling; b) associated to local buckling

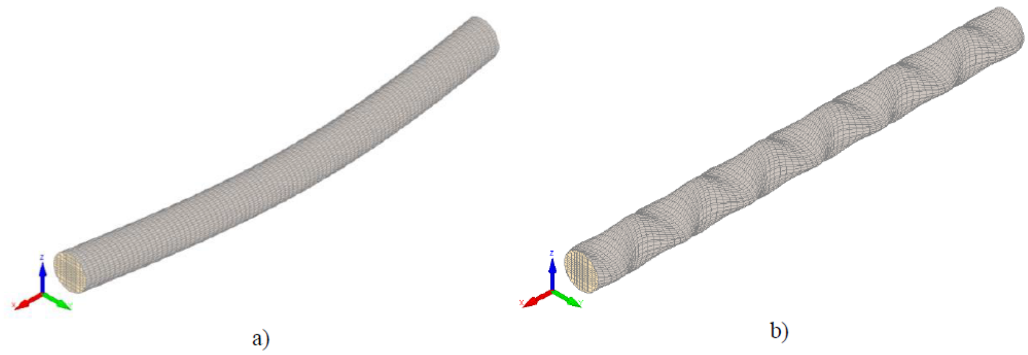


Fig. 5 Initial geometric imperfections on the square hollow profiles: a) associated to global buckling; b) associated to local buckling.

Residual stresses

In this work, longitudinal residual stresses due to fabrication processes were only considered in square hollow profiles. These residual stresses have low significance in the circular hollow profiles. Based on the model proposed by Gardner and Cruise [21], for cold formed sections the adopted residual stresses follow the distribution presented on Table 4.

Table 4 Residual stresses for the square hollow sections [21]

	Bending residual stresses	Membrane residual stresses
Central part of the plate	$\pm 0.63\sigma_{0.2}$	$+0.37\sigma_{0.2}$
External part of the plate	$\pm 0.63\sigma_{0.2}$	$-0.24\sigma_{0.2}$
Corners	$\pm 0.37\sigma_{0.2}$	$-0.24\sigma_{0.2}$

Corner strength enhancement

The cold formed manufacturing process as a positive influence on corners strength of rectangular hollow sections, which improves the cross-section resistance. Therefore on the numerical models, of the square hollow profiles, the corners regions were considered with strength enhancement following Ashraf et al. [22] studies. The corners regions were considered to spread from the corner for a distance of twice the plate thickness. The proportional limit strength on these corners is given by.

$$\sigma_{0.2,c} = \frac{1.881\sigma_{0.2,v}}{\left({}^{r_i}/_{t}\right)^{0.194}} \tag{16}$$

Where $\sigma_{0.2,v}$ is the proportional limit strength in the flat region of the plate.

And the ultimate strength in the corners region given by

$$\sigma_{u,c} = 0.75\sigma_{0.2,c}\frac{\sigma_{u,v}}{\sigma_{0.2,v}} \tag{17}$$

Where $\sigma_{u,v}$ is the ultimate strength in the flat region of the plate.

4 Parametric Study

The behaviour of the interaction curves N-M, described in Section 2, when compared to the numerical results in stainless steel beam-columns with hollow sections is presented. As mentioned in Section 2, these comparisons influence the flexural buckling curve on the interaction curves was eliminated considering the numerical results of resistance to axial compression on those interaction curves. As acknowledged in other research works [8,9], the results obtained with bending moment equal to 0 (pure compression), these buckling curves need to be improved for being too conservative. Moreover, following EC3 prescriptions, the calculation of the bending moment resistance does not consider the stainless steel hardening, which influences the approximation of the interaction curves to the numerical results.

The interaction curves presented in the comparisons are named of:

- "EN 1993-1-2" for the prescribed design rules in Part 1-2 of EC3 [3];
- "EN 1993-1-4 adapted" for the formulae in Part 1-4 of EC3 [6] adapted at elevated temperatures;
- "EN 1993-1-1adapted" for the formulae in Part 1-1 of EC3 [5] adapted to stainless steel at elevated temperatures;

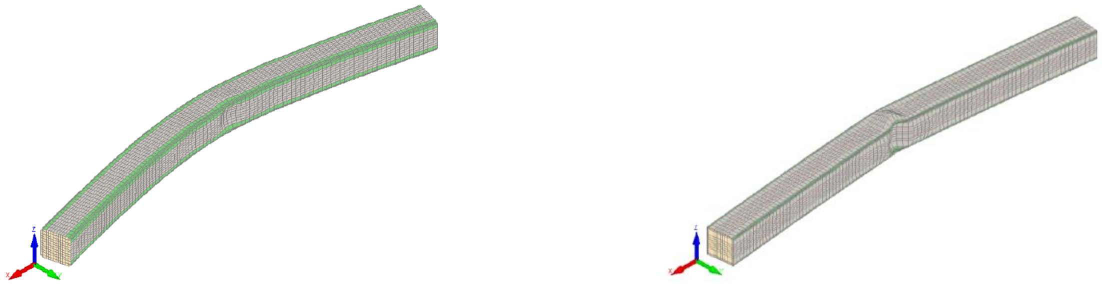
The cross section resistance is considered in all the obtained curves, applying the formulation presented in Section 2.1

4.1 Comparison with interaction curves

In this section some of the graphs resulting from the comparisons between the interaction curves and the numerical results are presented.

Beam-columns with square hollow sections

Fig. 6 presents examples of the deformed shape at failure of the beam-columns with square hollow sections of Class 2 where flexural buckling is predominant and of Class 4 where it is visible the local buckling occurrence.



a) b)

Fig. 6 Examples of the deformed shape at failure of the beam-columns with square hollow sections: a) Class 2 section; b) Class 4 section

For a better comparison of the influence of the different parameters it is first presented in Fig. 7 the results for beam-columns of 3 m length with Class 2 sections subjected to uniform bending at 500 ºC, and then these parameters are varied being analysed the behaviour and approximation of the different curves to the numerical results. It can be observed that for this case Part 1-4 of EC3 is not on safe side, whereas the other proposals provide good and safe approximations.

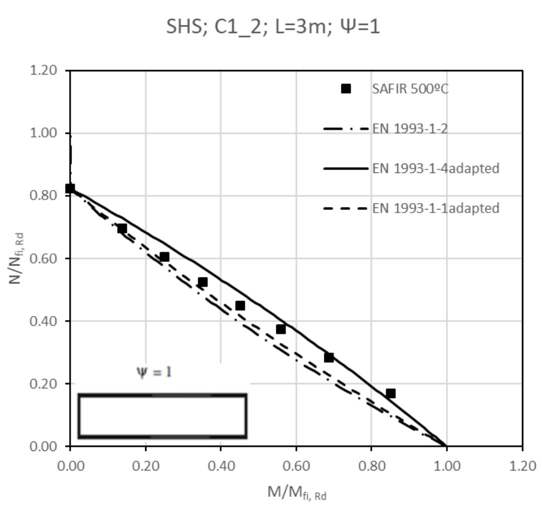
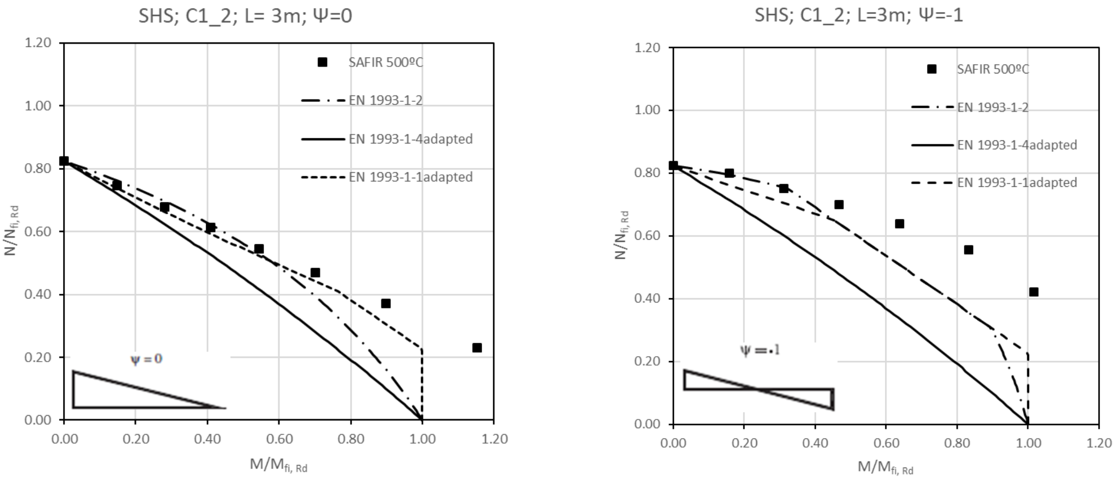


Fig. 7 Comparison of the interaction curves with the numerical results for beam-columns of 3 m length with Class 2 sections subjected to uniform bending at 500 ºC

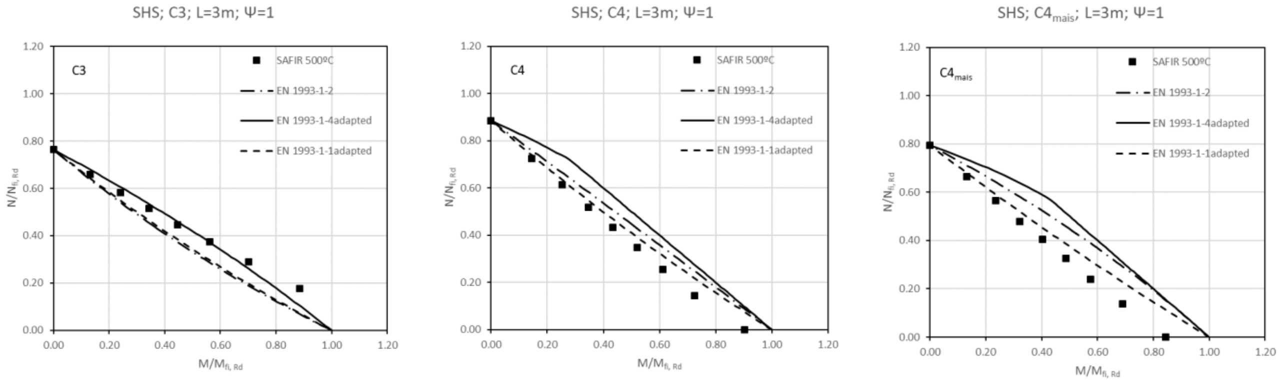
Varying the bending diagram shape it is visible on the graphs of Fig. 8 the loading type has big influence that should be considered by the interaction curves. As presented in Section 2 Part 1-4 of EC3 does not considers this influence being thus too conservative.



a) b)

Fig. 8 Comparison of the interaction curves with the numerical results for beam-columns of 3 m length with Class 2 sections subjected to bending diagrams at 500 ºC

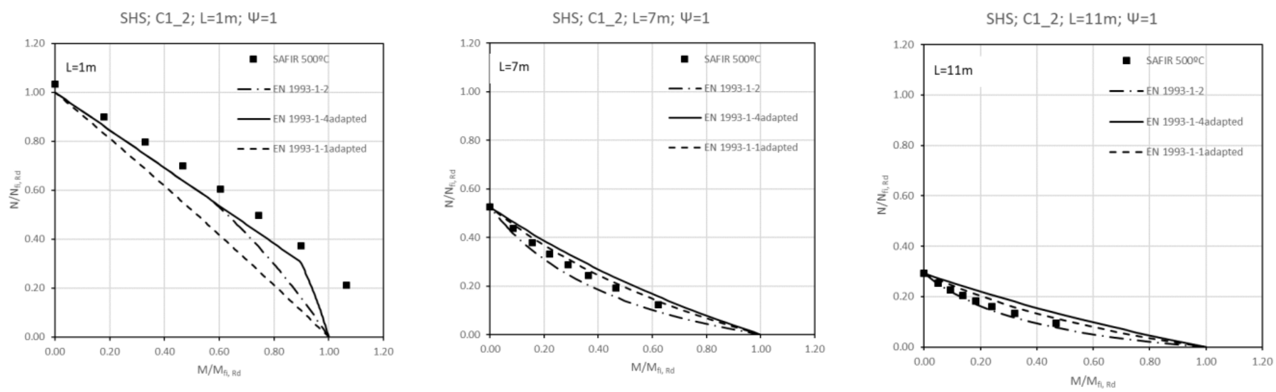
When sections with high slenderness (Fig. 9) are considered the interaction curves tend to be less conservative, which suggests that this parameter should be better considered. A more linear behaviour is observed on the numerical results.



a) b) c)

Fig. 9 Comparison of the interaction curves with the numerical results for beam-columns of 3 m length with cross-sections of different Classes subjected to uniform bending at 500 ºC

One important parameter is also the member global slenderness. Fig. 10 presents the evolution of the interaction curves when the member length is increasing. It can be observed that the safety degree of the interaction curves tend to decrease for longest beam-columns. Again Part 1-4 of EC3 presents the worst approximation to the numerical results and Part 1-2 of EC3 the best.

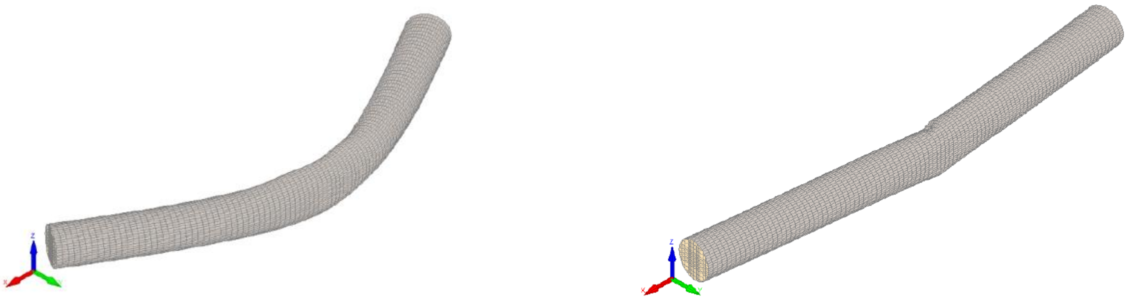


a) b) c)

Fig. 10 Comparison of the interaction curves with the numerical results for beam-columns of different lengths with Class 2 sections subjected to uniform bending at 500 ºC

Beam-columns with circular hollow sections

Similar analysis to the one presented for square hollow sections was also made for circular hollow sections. Fig. 11 presents examples of the deformed shape at failure of the beam-columns with circular hollow sections of Class 1 where flexural buckling is predominant and of Class 4 where it is visible the local buckling occurrence.



a) b)

Fig. 11 Examples of the deformed shape at failure of the beam-columns with circular hollow sections: a) Class 1 section; b) Class 4 section

Fig. 12 shows the results for beam-columns of 3 m length with Class 1 sections subjected to uniform bending at 500 ºC. All methodologies present safe results having a similar behaviour to the one observed for square hollow sections (Fig. 7).

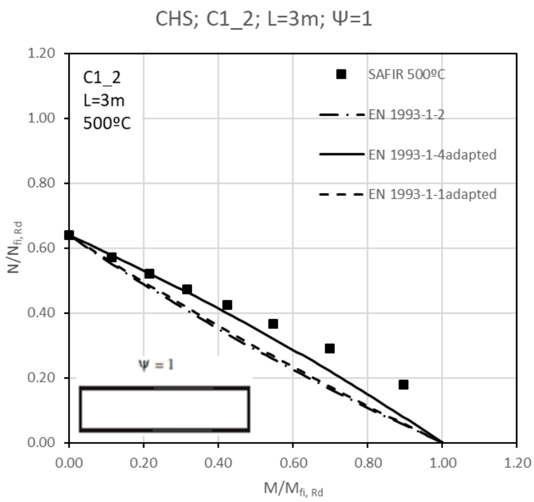
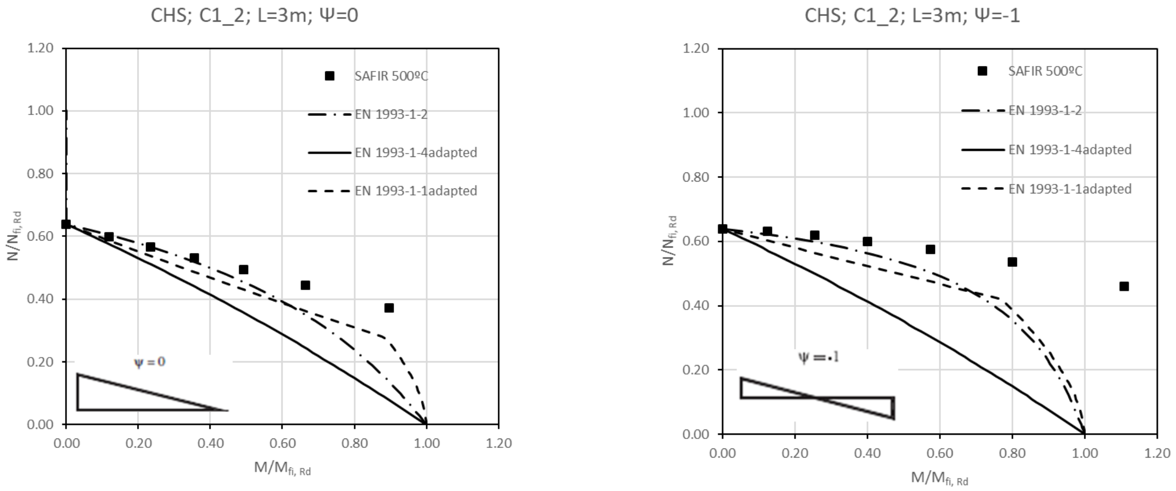


Fig. 12 Comparison of the interaction curves with the numerical results for beam-columns of 3 m length with Class 1 sections subjected to uniform bending at 500 ºC

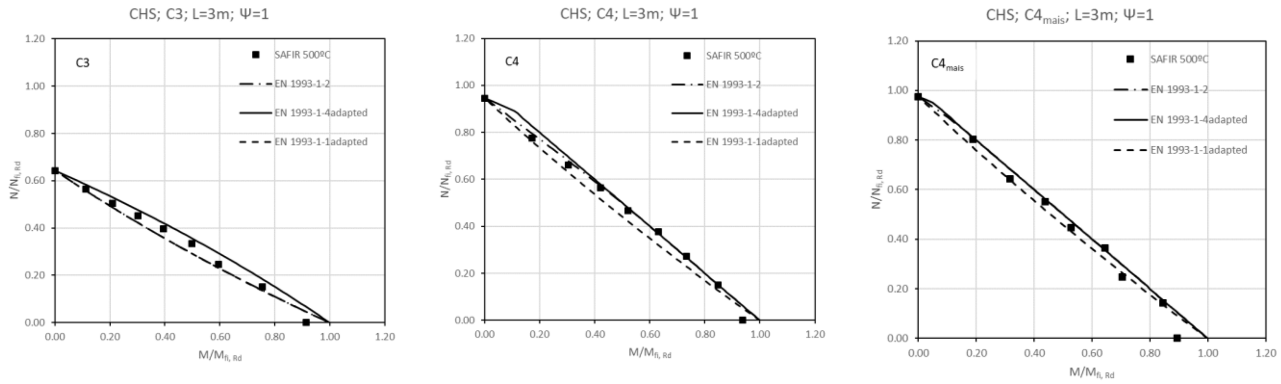
As expected the loading type has a big influence in the results, as it can be seen in Fig. 13.



a) b)

Fig. 13 Comparison of the interaction curves with the numerical results for beam-columns of 3 m length with Class 1 sections subjected to bending diagrams at 500 ºC

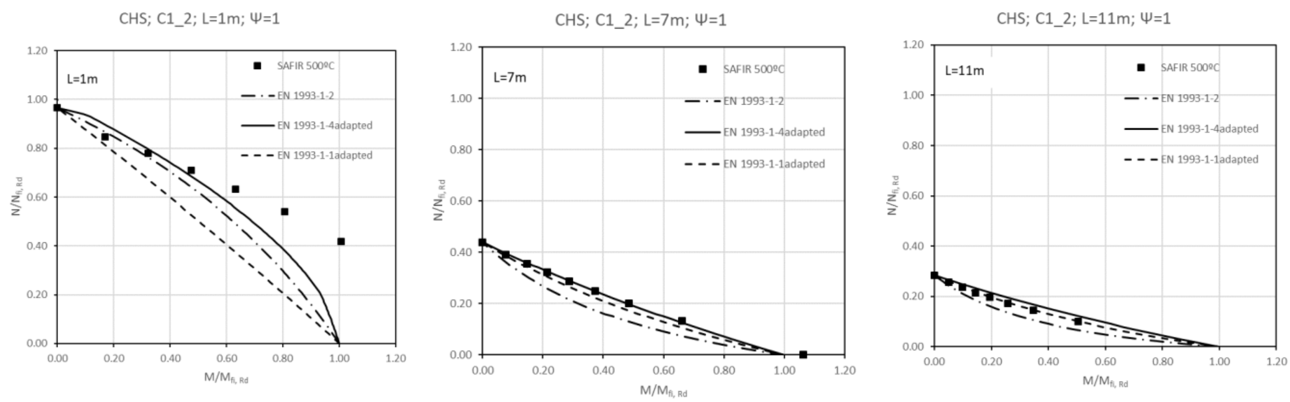
Contrary to what was observed for square hollow sections, the cross-section slenderness seems to be well considered by the different methodologies (Fig. 14).



a) b) c)

Fig. 14 Comparison of the interaction curves with the numerical results for beam-columns of 3 m length with cross-sections of different Classes subjected to uniform bending at 500 ºC

The member length is again an important parameter as shown in Fig. 15. It can be observed that Part 1-2 of EC3 is too conservative.



a) b) c)

Fig. 15 Comparison of the interaction curves with the numerical results for beam-columns of different lengths with Class 1 sections subjected to uniform bending at 500 ºC

4.2 Statistical evaluation of the different calculation methodologies

For a better evaluation of the approximations provided by the different methodologies, a brief statistical analysis is made here. Table 5 presents the statistical results for square hollow sections and Table 6 shows the statistical evaluation for circular Hollow sections.

Based on the evaluation criteria of Kruppa, J. [23], a given analytical formulation for fire design is considered safe if the comparison to the numerical results provides the following analyses. These plus the standard deviation are the calculation presented in tables 5 and 6.

- The average of the ratio between the numerical result and the analytical result is on the safe side (lower than 1.00);
- The percentage of the number of unsafe results is lower 20%;
- The maximum unsafe result provides a ratio lower of 1.5.

Table 5 Statistical results for square hollow sections

		Class 1 or 2 sections			Class 3 sections			Class 4 sections			
		ψ=1	ψ=0	ψ=-1	ψ=1	ψ=0	ψ=-1	ψ=1	ψ=0	ψ=-1	Global
Number of results		84	84	84	84	84	84	168	168	168	1008
EN1993-1-2	Average	0.93	0.91	0.90	0.91	0.90	0.90	1.10	1.03	1.08	0.99
	Standard deviation	0.03	0.08	0.09	0.07	0.09	0.09	0.10	0.13	0.21	0.15
	Unsafe results (%)	0	17	2	8	12	1	79	62	55	36
	Maximum unsafe	-	1.02	1.00	1.03	1.04	1.01	1.45	1.53	2.00	2.00
EN1993-1-4 adapted	média	1.01	0.85	0.73	0.98	0.85	0.75	1.18	0.98	0.94	0.95
	desvioP	0.05	0.08	0.10	0.07	0.08	0.12	0.11	0.12	0.14	0.17
	unsafe %	71	0	0	55	4	0	99	48	26	40
	max unsafe	1.07	-	-	1.08	1.03	-	1.62	1.40	1.39	1.62
EN1993-1-1 adapted	média	0.99	0.96	0.87	0.96	0.95	0.86	1.13	1.07	1.06	1.01
	desvioP	0.09	0.09	0.06	0.09	0.10	0.07	0.12	0.16	0.20	0.16
	unsafe %	50	50	0	42	43	0	98	79	46	53
	max unsafe	1.11	1.07	-	1.10	1.06	-	1.58	1.90	2.02	2.02

Table 6 Statistical results for circular hollow sections

		Class 1 or 2 sections			Class 3 sections			Class 4 sections			
		ψ=1	ψ=0	ψ=-1	ψ=1	ψ=0	ψ=-1	ψ=1	ψ=0	ψ=-1	Global
Number of results		84	84	84	84	84	84	84	84	84	756
EN1993-1-2	Average	0.88	0.87	0.88	1.03	1.05	1.15	0.89	0.83	0.86	0.94
	Standard deviation	0.06	0.10	0.11	0.08	0.09	0.21	0.12	0.11	0.06	0.15
	Unsafe results (%)	4	11	0	50	75	79	26	1	0	27
	Maximum unsafe	1.02	1.02	-	1.42	1.45	2.00	1.09	1.01	-	2.00
EN1993-1-4 adapted	média	0.96	0.82	0.72	1.11	0.99	0.95	0.90	0.83	0.86	0.91
	desvioP	0.06	0.10	0.13	0.10	0.07	0.09	0.13	0.11	0.06	0.14
	unsafe %	18	4	0	92	32	26	32	1	0	23
	max unsafe	1.07	1.02	-	1.61	1.29	1.19	1.09	1.01	-	1.61
EN1993-1-1 adapted	média	0.91	0.91	0.84	1.08	1.09	1.10	0.87	0.83	0.86	0.94
	desvioP	0.09	0.10	0.09	0.17	0.17	0.19	0.11	0.11	0.06	0.17
	unsafe %	21	0	0	50	61	64	8	1	0	23
	max unsafe	1.01	-	-	1.74	1.80	1.92	1.06	1.01	-	1.92

5 Conclusions

In this paper a numerical study on the behavior of stainless steel members with square and circular hollow sections in fire subjected to compression plus bending was presented. Comparisons between the obtained ultimate bearing loads, with the finite element program SAFIR, and different methods for the determination of the interaction curves N-M were performed, concluding that:

- The design prescriptions of Part 1-2 of EC3 do not provide safe approximations for Class 4 square hollow sections and for Class 3 circular hollow sections;
- The formulae from Part 1-4 of EC3 adapted to elevated temperatures give too conservative results when non-uniform bending is considered;
- The interaction curves from Part 1-1 of EC3, also adapted to elevated temperatures, gives unsafe approximations mainly for square hollow sections and also for Class 3 circular hollow sections.

Regarding the cross-section type, beam-columns with circular hollow sections provided more disperse numerical results when compared with the square hollow sections. However, the different design methodologies are generally safer for the members with circular hollow sections. When the cross-section classification is analysed, the different formulation provide unsafe approximations for Class 4 square hollow sections and Class 3 circular hollow sections.

It is clear that more studies are needed to better assess the behavior of the interaction between axial compression and bending moments in these profiles, being expected the development of new design formulae to provide safer and more accurate approximations to the ultimate bearing capacities of square and circular hollow profiles.

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