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Numerical analysis of stainless steel beam-columns in case of fire

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ABSTRACT

This paper presents an evaluation of the accuracy and safety of the design rules from Eurocode 3, for the fire resistance of stainless steel beam-columns, with and without lateral-torsional buckling. These evaluations are carried out performing an extensive numerical parametric study on beam-columns with equivalent welded H-cross-sections. In this study the influences of the residual stresses, the cross-sectional slenderness, the shape of the bending moment diagrams and the stainless steel grade are considered.

New proposals for the design equations of stainless steel beam-columns in case of fire are presented. These proposals revealed to be safer than the design equations of Eurocode 3 when compared with numerical results.

Additionally, the lateral-torsional buckling of stainless steel beams subjected to combined end moments and transverse loads in case of fire is also studied.

1. Introduction

The use of stainless steel in construction is increasing. However, it is still necessary to develop the knowledge of its structural behaviour. The most important advantage of stainless steels is their corrosion resistance, nevertheless, their aesthetic appearance, ease of maintenance, durability and the low life-cycle costs are also valuable characteristics. Experience has shown that the benefits of a long life with low maintenance and repair requirements more than compensates the higher initial purchase cost of stainless steel [1–3]. In addition, recent studies pointed to a good performance of stainless steel structures in fire situation, when compared against conventional carbon steel [4], in fact the stainless steel exhibits better thermal and mechanical properties when subjected to high temperatures [5, 6].

Part 1–4 of Eurocode 3 (EC3), "Supplementary rules for stainless steels" [7], gives design rules for stainless steel structural members at room temperature, and makes mention to its fire resistance by referring to the fire part of this Eurocode, EN 1993-1-2 [6]. As an example Fig. 1 presents a comparison between the stress–strain relationship of stainless steel 1.4301 and carbon steel S235 at 600 °C, according to the constitutive law formulae given in Part 1–2 of EC3 [6]. In this figure it can be also observed that the stainless steel exhibits a non-linear stress–strain relationship with an extensive hardening phase. Recent studies [8, 9] have evaluated the stainless steel constitutive laws at high temperatures and proposed different formulae, however as this work aims at evaluating Eurocode accuracy, the EC3 stress–strain relationship have been followed. Fig. 2 compares the yield strength reduction factor at high temperatures relative to the value of the yield strength at 20 °C for carbon steel and different stainless steel grades ($k_{y,\theta}=f_{y,\theta}/f_y$) [6]. From these figures it can be concluded that the mechanical behaviours of structural members on carbon steel and stainless steel, in case of fire, can be rather different, which is not the assumption made in EN 1993-1-2, where it is stated that stainless steel structural members subjected to high temperatures must be designed with the same formulae as those used for carbon steel members. As these two materials have different constitutive laws at elevated temperature, it can be expected that different formulae for the member stability calculation should be used for fire design, as it is the case for normal temperature design. In fact EN 1993-1-4 (for cold design of stainless steel) and EN 1993-1-1 [10] (for cold design of carbon steel) present different formulae for checking the resistance of structural members at normal temperature. Some authors have already recognised the need of using different formulations and, for example, new design models for stainless steel hollow columns in case of fire based in both experimental and numerical tests have been recently proposed [11, 12].

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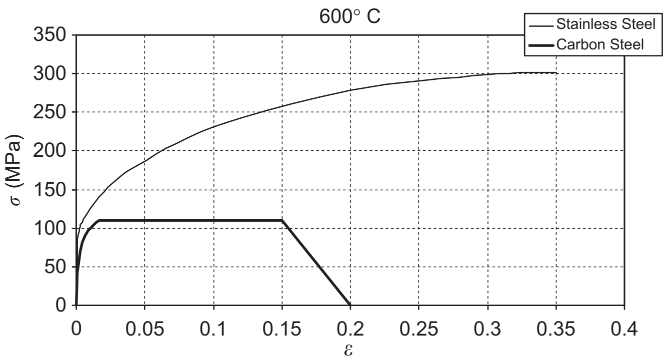

Fig. 1. Stress–strain relationships of carbon steel S 235 and stainless steel 1.4301 at 600 °C.

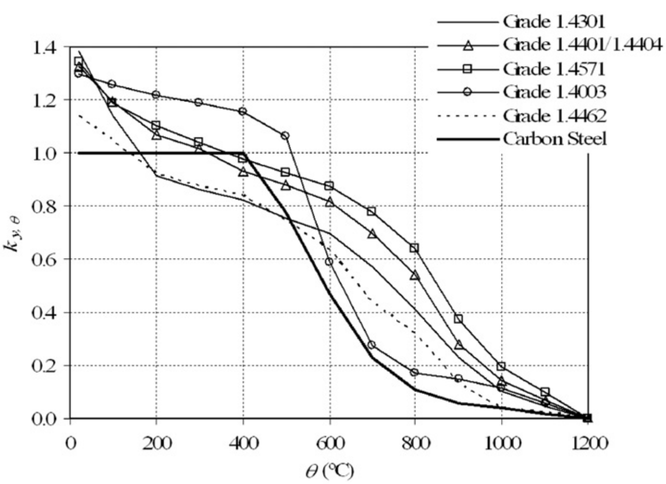

Fig. 2. Reduction factor of the yield strength of carbon steel and stainless steel.

The main objective of this work is to evaluate the accuracy and safety of the currently prescribed design rules from EC3, for the fire resistance of stainless steel uniform beam-columns, with and without lateral-torsional buckling (LTB). An extensive campaign of parametric numerical simulations has been performed on Class 1 and Class 2 stainless steel equivalent welded H-cross-sections. Comparisons between numerical simulations and results obtained with the design equations of EC3 for those elements are presented. Based on these comparisons, safer and more accurate proposals for the interaction curves of stainless steel beam-columns are presented, taking into account the influence of the residual stresses, the cross-sectional slenderness, shape of the bending diagrams and the stainless steel grade.

In previous works by the authors [13, 14], new formulae for the flexural buckling of stainless steel columns and for the LTB of stainless steel beams in case of fire were proposed. These new proposals necessarily affect the interaction formulae for stainless steel beam-columns in case of fire and therefore their influence were taken into account in the present work. As in the reference [14] the authors did not treat the case of combined end moments and transverse loads, in the present paper, new correction factors for the LTB safety evaluation of stainless steel members in case of fire, which improve the EC3 design buckling curve for this type of load, is also presented.

For beam-columns in case of fire, Part 1-2 of EC3 adopts the format given in Part 1-1 of EC3 from 1992 [15], replacing: (i) the yield strength and the Young's modulus with their values at high temperatures; (ii) the buckling reduction factors for flexural buckling and LTB by the values recommended for carbon steel in case of fire; and also, when no LTB occurs, (iii) the interaction curves by the curves proposed by Talamona et al. [16, 17]. The final version of Part 1-1 of EC3, EN 1993-1-1 [10], introduced several changes in the design formulae for carbon steel beam-columns at normal temperature when compared with the ENV version of EC3 [15]. Two new methods, which are the result of the work carried out by two working groups who followed different approaches [18–20], are proposed in the EN version of Part 1-1 of EC3. Some studies [21–24] have been carried out to check if these new design approaches for normal temperature can also be used in case of fire, which concluded that additional changes should be introduced. In the present study these new formulae were also tested, after being adapted to deal with stainless steel in fire situation.

Although, at normal temperature the EC3 simple formulae for the evaluation of stainless steel members resistance are based on the 0.2% proof strength, $f_y = f_{0.2\text{proof}}$, Part 1-2 of EC3 suggests for fire design the use of the stress at 2% total strain for yield strength at elevated temperature for elements with Class 1, 2 and 3 cross-sections In the simple formulae proposed in this paper, the use of the stress at 2% total strain for the yield strength at elevated temperature θ, $f_{y,\theta} = f_{2,\theta}$, is considered, preserving the same philosophy used in the Eurocode.

2. Numerical model

The numerical results were obtained with the programme SAFIR [25, 26] using the methodology usually designated by GMNIA (geometrically and materially non-linear imperfect analysis). To model the behaviour of stainless steel structures, the programme has been adapted according to the stress–strain relationship defined in Annex C of Part 1-2 of EC3 [6] (see as an example Fig. 1 for the grade 1.4301 at 600 °C). The stainless steel mechanical properties at high temperatures have been introduced in a one-dimensional constitutive model to be used on the bar and beam finite elements. Validation of the programme has been made by comparisons against other software numerical results and experimental tests [27].

A parametric study of the behaviour of stainless steel beam-columns in different grades (austenitic, austenitic–ferritic and ferritic grades), under different loading and subjected to fire is presented. The equivalent welded European cross-sections HE200A in stainless steel grade 1.4301 (austenitic), HE280B in stainless steel grades 1.4301 and 1.4003 (ferritic), and HE200B in stainless steel grade 1.4462 (duplex) were used. These profiles were chosen to be representative of the Class 1 or Class 2 H-sections. An average of 5 beam lengths and 8 bending moment/axial load ratios were analysed for each case, resulting in a total of 961 numerical simulations. A uniform temperature distribution of 600 °C in the cross-section was used, due to being a common critical temperature in steel members.

In the numerical simulations, a sinusoidal lateral geometric imperfection was considered [28].

$$y(x) = \frac{L}{1000}\sin\left(\frac{\pi x}{L}\right) \tag{1}$$

where L is the length of the structural element. This global imperfection was imposed in function of the study case in the plane $x0y$ or in the plane $x0z$.

The lengths of the beam-columns analysed in this work (1, 3, 5, 7, 10 and 14 m) were chosen in order to have non-dimensional slenderness values up to two.

Two types of finite elements were used: a 2D finite beam element for the cases where LTB was not considered and a 3D one for the cases where LTB has been studied. The beam-columns