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Lateral–torsional buckling of stainless steel I-beams in case of fire

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A B S T R A C T

This work presents a numerical study of the behaviour of stainless steel I-beams subjected to lateral–torsional buckling in case of fire and compares the obtained results with the beam design curves of Eurocode 3.

New formulae for lateral–torsional buckling, that approximate better the real behaviour of stainless steel structural elements in case of fire are proposed. These new formulae were based on numerical simulations using the program SAFIR, which was modified to take into account the material properties of the stainless steel.

1. Introduction

There are five basic groups of stainless steels, classified according to their metallurgical structure: the austenitic, ferritic, martensitic, duplex austenitic–ferritic and precipitation-hardening groups [1]. Austenitic stainless steels provide a good combination of corrosion resistance, forming and fabrication properties. Duplex stainless steels have high strength and wear resistance with very good resistance to stress corrosion cracking. The most commonly used grades, typically referred to as the standard austenitic grades, are 1.4301 (widely known as 304) and 1.4401 (widely known as 316). The austenitic stainless steels are generally the most used groups for structural applications but some interest is being recently shown for increasing the use of ferritic steels for structural purposes, due to their relative lower cost.

The use of stainless steel for structural purposes has been limited to projects with high architectural value, where the innovative character of the adopted solutions is a valorisation factor for the structure. The high initial cost of stainless steel, coupled with: (i) limited design rules, (ii) reduced number of available sections and (iii) lack of knowledge of the additional benefits of its use as a structural material, are some of the reasons that force the designers to avoid its use [2,3]. However, a more accurate analysis shows a good performance of stainless steel when compared with conventional carbon steel.

Part 1.4 of Eurocode 3 "Supplementary rules for stainless steels" [4] gives design rules for stainless steel structural elements at room temperature, and only mentions its fire resistance by referring to the fire part of the same Eurocode, EN 1993-1-2 [5]. Although carbon steel and stainless steel have different constitutive laws, Eurocode 3 states that the structural elements made of these two materials must be checked for its fire resistance using the same formulae. Fig. 1 shows a comparison between the nominal stress–strain relationships of carbon steel S235 and stainless steel 1.4301 at 600 °C.

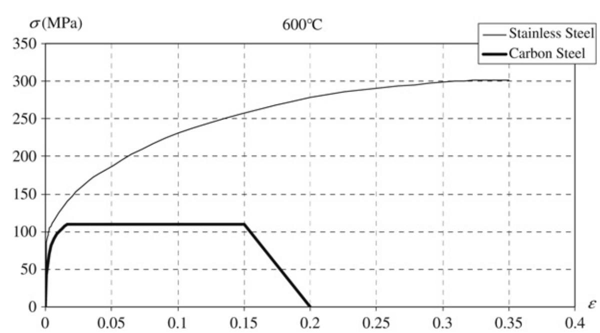

Fig. 1. Stress–strain relationships of carbon steel S 235 and stainless steel 1.4301 at 600 °C.

Stainless steels are known for their nonlinear stress–strain relationships with a low proportional stress and an extensive hardening phase [6,7]. There is not a well defined yield strength, being usually considered for design at room temperature the 0.2% proof strength, $f_y = f_{0.2\text{proof}}$. In a fire situation higher strains than at room temperature are acceptable, and part 1.2 of Eurocode 3 suggests the use of the stress at 2% [8] total strain as the yield stress

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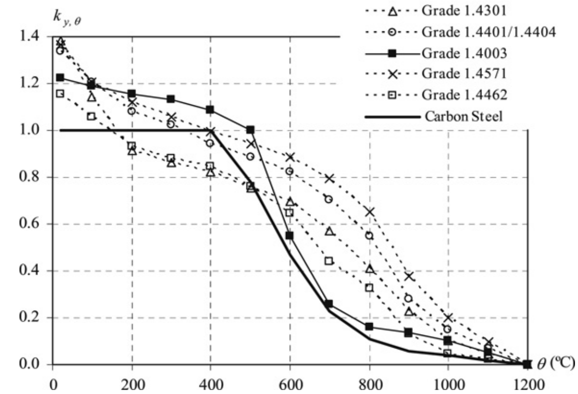


Fig. 2. Strength reduction at high temperatures.

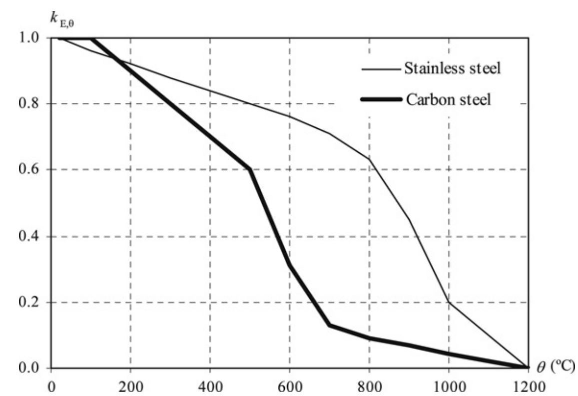


Fig. 3. Elastic stiffness reduction at high temperatures.

at elevated temperature θ, $f_{y,\theta} = f_{2,\theta}$, for Class 1, 2 and 3 cross-sections and $f_{y,\theta} = f_{0.2proof,\theta}$, for Class 4.

For the evaluation of the yield strength reduction factor, the Eurocode states that the following equation should be used:

$$k_{y,\theta} = \frac{f_{y,\theta}}{f_y} = \left[f_{0.2p,\theta} + k_{2\%,\theta}\left(f_{u,\theta} - f_{0.2p,\theta}\right)\right]\frac{1}{f_y}, \quad (1)$$

where $f_{0.2p,\theta}$ is the proof strength at 0.2% plastic strain, at temperature θ; $k_{2\%,\theta}$ is the correction factor for determination of the yield strength $f_{y,\theta}$; $f_{u,\theta}$ is the ultimate tensile strength, at temperature θ.

Comparison of the reduction of strength and elastic stiffness of structural carbon steel and stainless steel at elevated temperature for several grades of stainless steels, as defined in Eurocode 3, is shown in Figs. 2 and 3, where $k_{y,\theta} = f_{y,\theta}/f_y$ and $k_{E,\theta} = E_\theta/E$, $f_{y,\theta}$ and f_y are the yield strength at elevated temperature and at room temperature respectively, and E_θ and E are the modulus of elasticity at elevated temperature and at room temperature.

In this paper a new proposal for the lateral–torsional buckling of stainless steel beams, different from the formulae for carbon steel, will be made.

From Fig. 2 it can be also observed that, according the Eurocode 3 [5], the variation of the strength reduction, of the stainless steel grade 1.4003 (the only ferritic stainless steel grade referred in part 1.2 of Eurocode 3) with temperature is different from the other stainless steel grades, mainly for the temperature range between 500 and 700 °C. The reduction of the yield strength and of the elasticity's modulus are used in the determination of the non-dimensional slenderness at high temperatures, as it will be shown later in this work. This fact affects the behaviour of unrestrained 1.4003 stainless steel beams and suggests that the stainless steel grade should also be taken into account in the design of unrestrained beams.

The lateral–torsional buckling curves proposed in the ENV version of part 1.1 of Eurocode 3 [9] (carbon steel design at room temperature) only took in consideration the loading type in the determination of the elastic critical moment, not accounting for the additional beneficial effect resulting from the reduction of the plastic zones, directly related to the fact that the bending diagrams are variable along the beam, leading to over-conservative results in beams not subjected to uniform bending diagrams [10]. As for other international regulations [11,12], where this effect was already considered, a correction factor that considers the loading type was introduced in EN 1993-1-1 [13]. This effect still remains to be taken into account in part 1.4 and part 1.2 of Eurocode 3.

Therefore, alternative expressions for carbon steel beams in case of fire and for stainless steel beams at room temperature with lateral–torsional buckling were proposed, ensuring the compatibility and coherence between part 1.1, part 1.2 and part 1.4 of Eurocode 3, as well as supplying a simple, competitive, and safe procedure. These new proposals [14,15] follow the same approach as in part 1.1 of Eurocode 3, also taking into consideration the influence of the loading type.

Codes of practice are aimed at providing safe, competitive and, as far as possible, simple procedures for the design of structures. Drafting and implementing a consistent set of structural Eurocodes involving a large number of groups of experts is naturally a recursive task where each part must reflect the scientific advances and design options of all other related parts.

The program SAFIR [16] has been used in the numerical simulations. This program is a geometrical and material nonlinear finite element code, specially developed in the University of Liege for the study of structures in case of fire, and it has been adapted, according to the material properties defined in part 1.4 [4] and part 1.2 [5] of Eurocode 3, to model the behaviour of stainless steel structures. This program, widely used by several investigators, has been validated against analytical solutions, experimental tests and numerical results from other programs, and has been used in several studies that lead to proposals for safety evaluation of structural elements, already adopted in Eurocode 3.

Comparisons between the numerical results obtained with the program SAFIR, and the buckling curves from part 1.2 of Eurocode 3, for unrestrained stainless steel beams in case of fire, will be presented. Based on these comparisons, a proposal for the lateral–torsional buckling resistance, safer and more accurate than the formulae from the Eurocode 3, is made.

2. Eurocode 3 formulae for the lateral–torsional buckling of stainless steel elements

For stainless steel beams subjected to elevated temperatures, part 1.4 of Eurocode 3 [4] refers that the same formulation prescribed for carbon steel elements must be used. According to the EN 1993-1-2 [5], the lateral–torsional buckling resistant moment for class 1 and class 2 cross-sections, is

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{pl,y} k_{y,\theta} f_y \frac{1}{\gamma_{M,fi}} \quad (2)$$

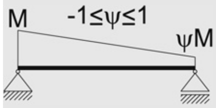
where $\chi_{LT,fi}$ is given by

$$\chi_{LT,fi} = \frac{1}{\phi_{LT,\theta} + \sqrt{\left(\phi_{LT,\theta}\right)^2 - \left(\bar{\lambda}_{LT,\theta}\right)^2}} \quad (3)$$

with

$$\phi_{LT,\theta} = \frac{1}{2}\left[1 + \alpha\bar{\lambda}_{LT,\theta} + \left(\bar{\lambda}_{LT,\theta}\right)^2\right]. \quad (4)$$

Table 1
Correction factors k_c for the new proposal

Bending diagram	Class 1, 2, 3 sections
	k_c
	$0.6 + 0.3\psi + 0.15\psi^2$ but $k_c \leq 1$
	0.79
	0.91

For others bending diagrams $k_c = 1$.

Table 2
Values of the severity factor β, for carbon steel

Cross-section	Limits	β	
		$\alpha = \beta\sqrt{\frac{235}{f_y}}$	
		S235, S275, S355, S420	S460
Welded I section	$h/b \leq 2$	0.70	0.75
	$h/b > 2$	0.80	0.85

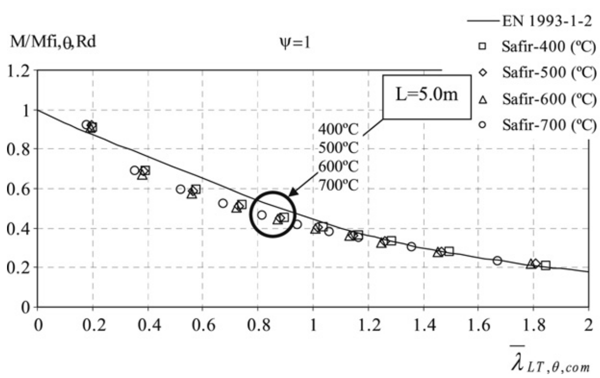


Fig. 4. Lateral–torsional buckling in IPE 500 beams of the stainless steel grade 1.4301.

In this expression the imperfection factor α depends on the steel grade and is determined with

$$\alpha = 0.65\varepsilon \tag{5}$$

where ε is given in part 1.1 of Eurocode 3 [13] as

$$\varepsilon = \sqrt{235/f_y}. \tag{6}$$

The imperfection factor is then given by

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

The non-dimensional slenderness for lateral–torsional buckling at high temperatures is given by

$$\bar{\lambda}_{LT,\theta} = \bar{\lambda}_{LT}\left[\frac{k_{y,\theta}}{k_{E,\theta}}\right]^{0.5} \tag{8}$$

where the reduction factor for the yield strength $k_{y,\theta}$, at temperature θ, is determined with Eq. (1) and $\bar{\lambda}_{LT}$ is the non-dimensional slenderness at room temperature.

3. Proposal for the lateral–torsional buckling of carbon steel elements

The authors have made a new proposal for the lateral–torsional buckling of carbon steel beam elements in case of fire [14] that adopts, following EN 1993-1-1 [13], a modified reduction factor for the lateral–torsional buckling $\chi_{LT,fi,\mathrm{mod}}$, given by

$$\chi_{LT,fi,\mathrm{mod}} = \frac{\chi_{LT,fi}}{f}, \quad \text{but } \chi_{LT,fi,\mathrm{mod}} \leq 1 \tag{9}$$

where f depends on the loading type and is determined by

$$f = 1 - 0.5\,(1 - k_c)\,. \tag{10}$$

The correction factor k_c is defined according to Table 1.

To take into account the cross-section type and the steel grade S460, the imperfection factor α given in Eq. (5), is written as a function of a severity factor β

$$\alpha = \beta\varepsilon \tag{11}$$

where, ε is given by Eq. (6), coming the imperfection factor as

$$\alpha = \beta\sqrt{235/f_y}. \tag{12}$$

For welded sections the severity factor is given in Table 2 [14]. The effect of this severity factor is to move the beam design curve in the vertical direction placing it closer to the numerical points.

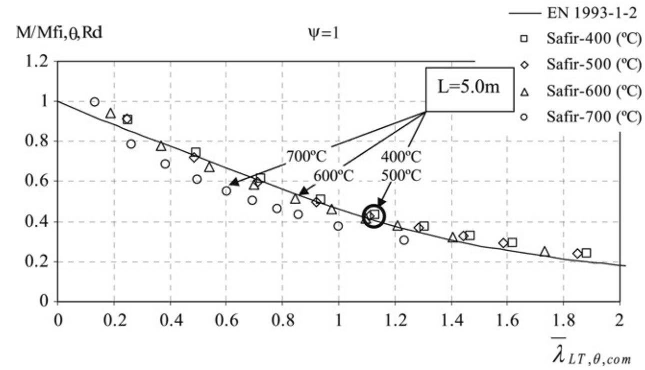


Fig. 5. Lateral–torsional buckling for in IPE500 beams of the stainless steel grade 1.4003.

4. Proposal for the lateral–torsional buckling of stainless steel elements

Based on the proposal made by the authors [14] for lateral–torsional buckling of carbon steel beams, described in the previous section, similar numerical studies of stainless steel beams subjected to high temperatures, were made. These studies also resulted in the proposal of Eqs. (9) and (10) (used for carbon steel), for unrestrained stainless steel beams in case of fire.

Figs. 4 and 5 compare the beam design curve from Eurocode 3 with the numerical results obtained with SAFIR. In the vertical axis, the relation between M, the resistant moment given by the Eurocode or by the program SAFIR and $M_{fi,\theta,Rd}$, the plastic resistant moment at temperature θ, given by:

$$M_{fi,\theta,Rd} = W_{pl,y}k_{y,\theta}f_y \tag{13}$$

is plotted.

Figs. 4 and 5 show, for uniform bending diagrams ($\psi = 1$), the curve resulting from the Eurocode 3 is not on the safe side, compared with the numerical values. For the ferritic stainless steel grade 1.4003, Fig. 5 shows that a beam with a length of 5 m exhibits slenderness values for 600 °C and 700 °C quite different from the slenderness values for 400 °C and 500 °C. These differences, due to the behaviour of the reduction factor for the yield strength function of the temperature, as shown in Fig. 2, are not as big for the austenitic stainless steel. In Eq. (8) the slenderness at room temperature is multiplied by the factor $\left(k_{y,\theta}/k_{E,\theta}\right)^{1/2}$ in order to obtain the slenderness at high temperatures. Fig. 8 shows that from

Table 3
New proposal for the severity factor β to be used with Eq. (15)

Cross-section	Limits	β $\alpha = \beta\sqrt{\frac{235}{f_y}\frac{E}{210\,000}}\sqrt{\frac{k_{E,\theta}}{k_{y,\theta}}}$	
		Austenitic and Duplex stainless steel	Ferritic stainless steel 1.4003
Welded I section	$h/b \le 2$	0.80	0.6
	$h/b > 2$	0.95	0.7

Table 4
New proposal for the severity factor β to be used with Eq. (12)

Cross-section	Limits	β $\alpha = \beta\sqrt{\frac{235}{f_y}}$	
		Austenitic and Duplex stainless steel	Ferritic stainless steel 1.4003
Welded I section	$h/b \le 2$	0.85	1.00
	$h/b > 2$	1.00	1.20

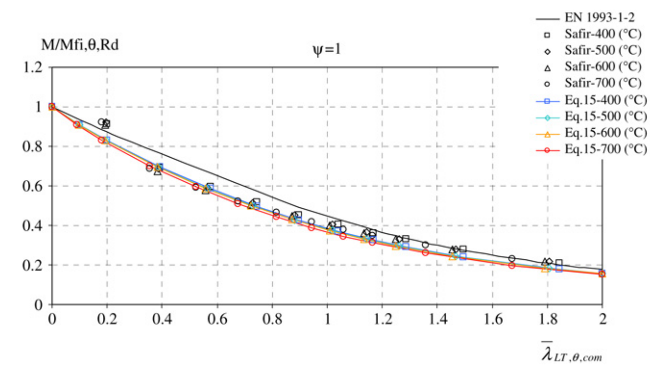


Fig. 6. Lateral–torsional buckling in IPE 500 beams of the stainless steel grade 1.4301. Curves obtained with Eq. (15).

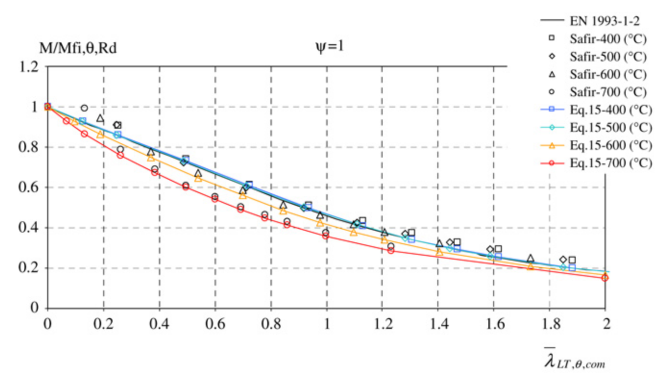


Fig. 7. Lateral–torsional buckling for in IPE500 beams of the stainless steel grade 1.4003. Curves obtained with Eq. (15).

500 to 700 °C, there is a great decrease of this factor for the 1.4003 stainless steel, which does not occur with the others stainless steel grades.

To improve the accuracy of the design curve from Eurocode 3, bringing it down, a new imperfection factor is used, based on Eq. (11) and using ε given in part 1.4 of the Eurocode 3 [4]

$$\varepsilon = \sqrt{\frac{235}{f_y}\frac{E}{210\,000}}. \tag{14}$$

This factor can be written as a function of the temperature, being the imperfection factor given by:

$$\alpha = \beta\sqrt{\frac{235}{f_y}\frac{E}{210\,000}}\sqrt{\frac{k_{E,\theta}}{k_{y,\theta}}}. \tag{15}$$

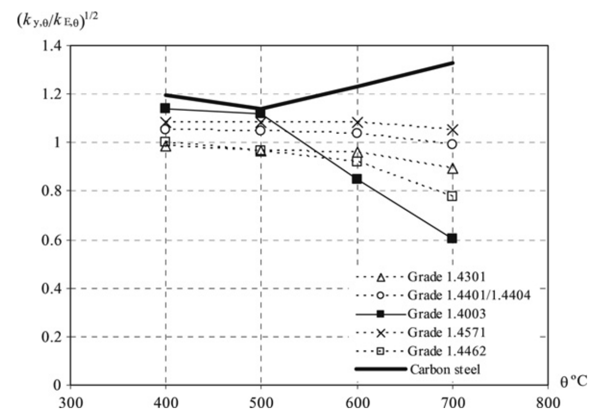


Fig. 8. Variation of the square root used in the determination of the slenderness.

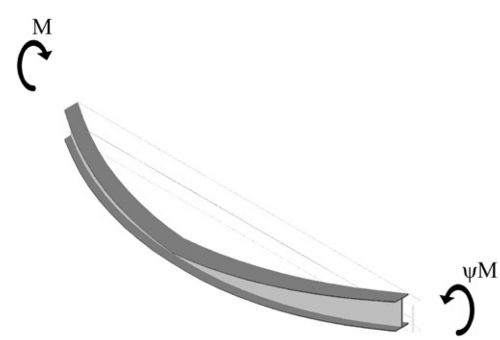


Fig. 9. Simply supported beam with non-uniform bending.

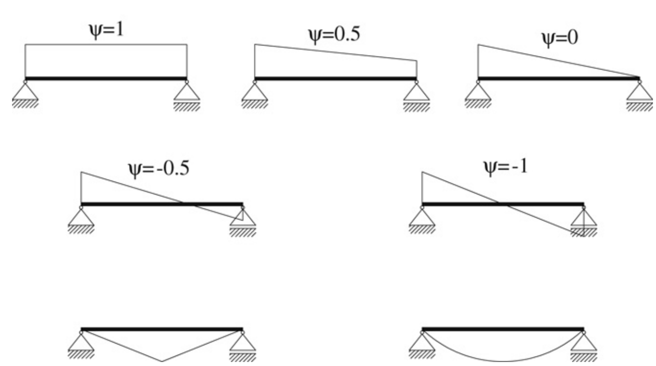


Fig. 10. Studied bending diagrams.

Table 3 gives the values of factor β to be used with Eq. (15) and Figs. 6 and 7 show the beam design curve obtained with this new imperfection factor.

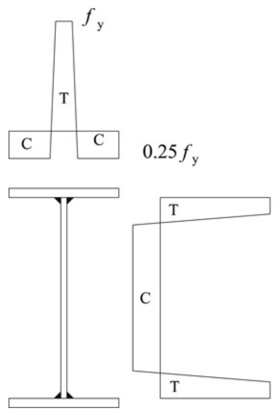


Fig. 11. Residual stresses: C—compression; T—tension.

To avoid the use of an imperfection factor depending on the temperature, it is proposed to use Eq. (12) with the severity factor defined in Table 4, called "New proposal" hereafter.

5. Parametric study

A simply supported beam with fork supports, as shown in Fig. 9, was chosen to explore the validity of the beam safety verifications. Regarding the bending moment variation along the member length, five values (−1, −0.5, 0, 0.5 and 1) of the ψ ratio have been investigated as well as a mid span concentrated load and a uniformly distributed load, as illustrated in Fig. 10.

The influence of the cross-sectional shape, assessed using the height/width (h/b) relation, was taken into account in this work. The following welded equivalent cross-sections were used: IPE 220 steel section (representative of $h/b = 2$), HEA 500 steel section (representative of $h/b < 2$) and IPE 500 steel section (representative of $h/b > 2$).

The stainless steel grades 1.4301, 1.4401, 1.4462, 1.4571 and the ferritic 1.4003, referred in part 1.2 of the Eurocode 3 [5], were studied for each cross-section. A uniform temperature distribution in the cross-section was used so that comparison between the numerical results and the Eurocode could be made. In this paper, the temperatures chosen were 400, 500, 600 and 700 °C, deemed to cover the majority of practical situations.

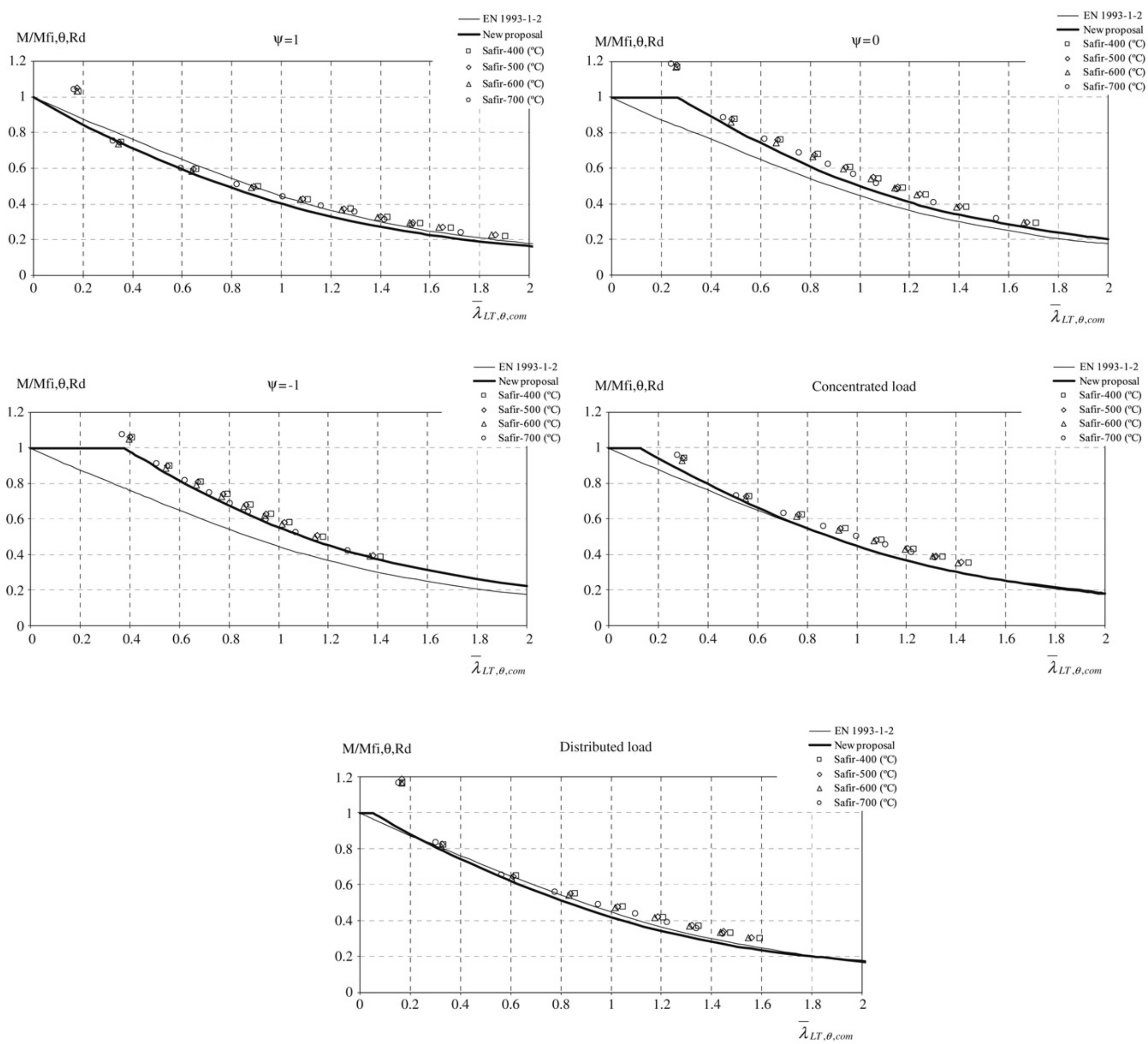


Fig. 12. Lateral–torsional buckling in IPE 220 beams in stainless steel 1.4301.

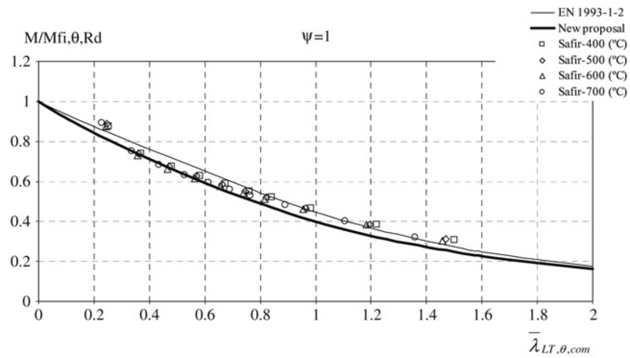


Fig. 13. Lateral–torsional buckling in HEA 500 beams in stainless steel 1.4301.

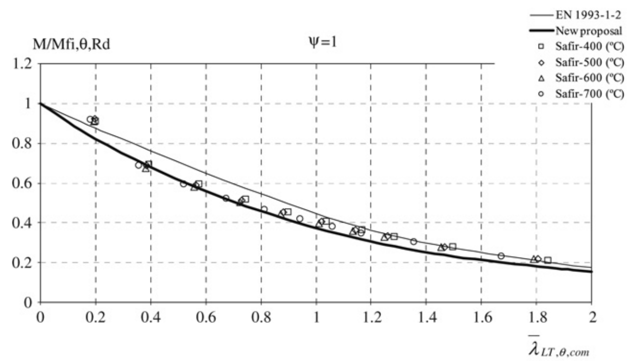


Fig. 14. Lateral–torsional buckling in IPE 500 beams in stainless steel 1.4301.

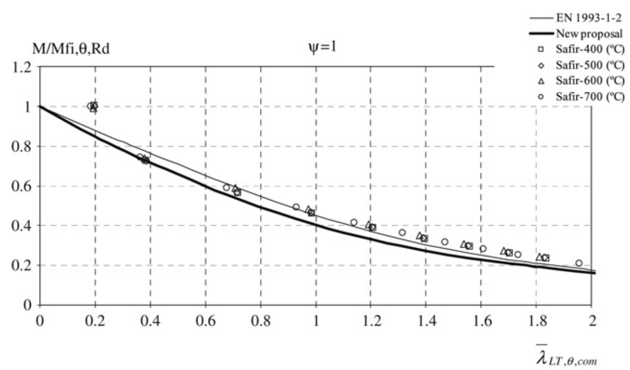


Fig. 15. Lateral–torsional buckling in IPE 220 beams in stainless steel 1.4401.

In the numerical simulations, a lateral geometric imperfection with a maximum value of $l/1000$ [17], given by the following expression was considered:

$$y(x) = \frac{l}{1000}\sin\left(\frac{\pi x}{l}\right) \tag{16}$$

where l is the length of the beam. An initial rotation around the beam axis with a maximum value of $l/1000$ rad at mid span was also considered.

The adopted residual stresses follow, the typical patterns for carbon steel welded sections [18–20], considered constant across the thickness of the web and flanges. The distribution is shown in Fig. 11, and has the maximum value of f_y (yield strength).

In this parametric study, Figs. 12–19 compare the curves obtained using part 1.4 of Eurocode 3, described in Section 2 of this paper (denoted "EN 1993-1-2"), the curve obtained with the proposal presented in Section 4 (denoted "New proposal"), and the numerical results obtained with the program SAFIR.

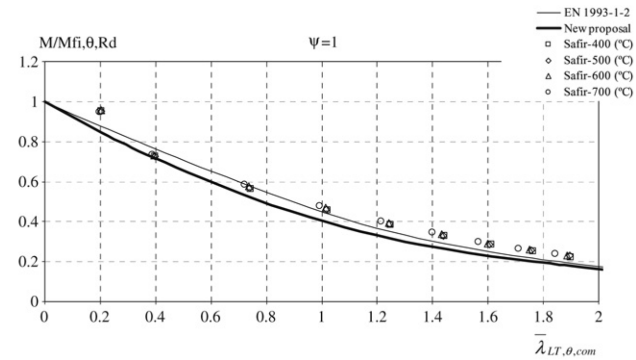


Fig. 16. Lateral–torsional buckling in IPE 220 beams in stainless steel 1.4571.

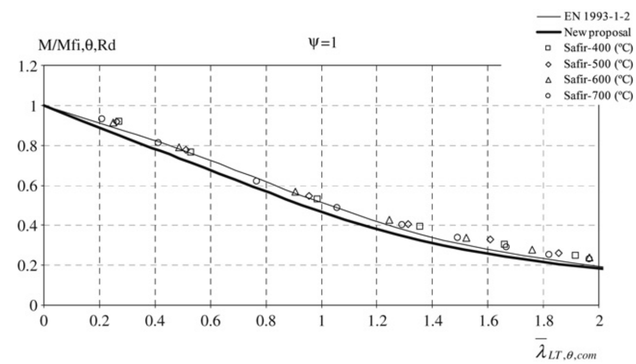


Fig. 17. Lateral–torsional buckling in IPE 220 beams in stainless steel 1.4462.

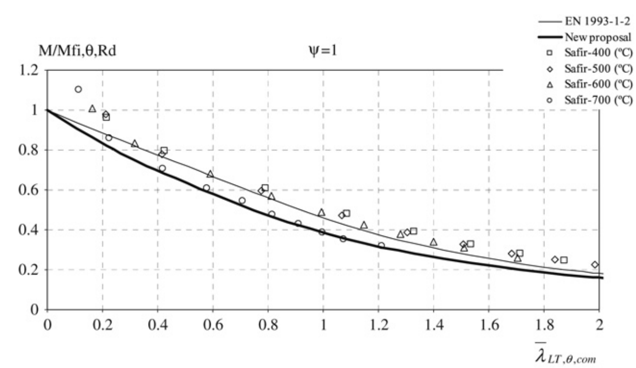


Fig. 18. Lateral–torsional buckling in IPE 220 beams in stainless steel 1.4003.

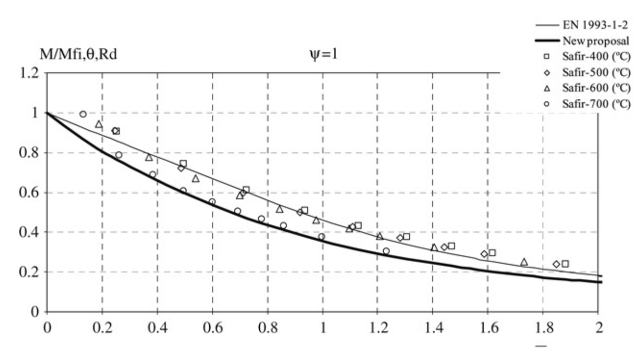


Fig. 19. Lateral–torsional buckling in IPE 500 beams in stainless steel 1.4003.

Results for IPE 220 beams, in stainless steel 1.4301, subjected to lateral–torsional buckling in case of fire are shown in Fig. 12, for three values, (−1, 0 and 1) of the ψ ratio, a mid span concentrated

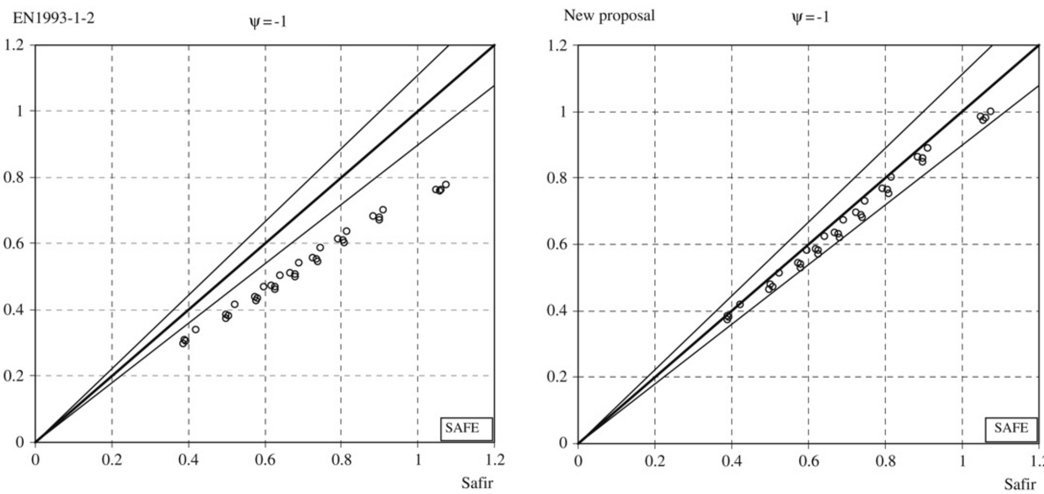


Fig. 20. Improvement for the case of $\psi = -1$ in IPE 220 beams in stainless steel 1.4301.

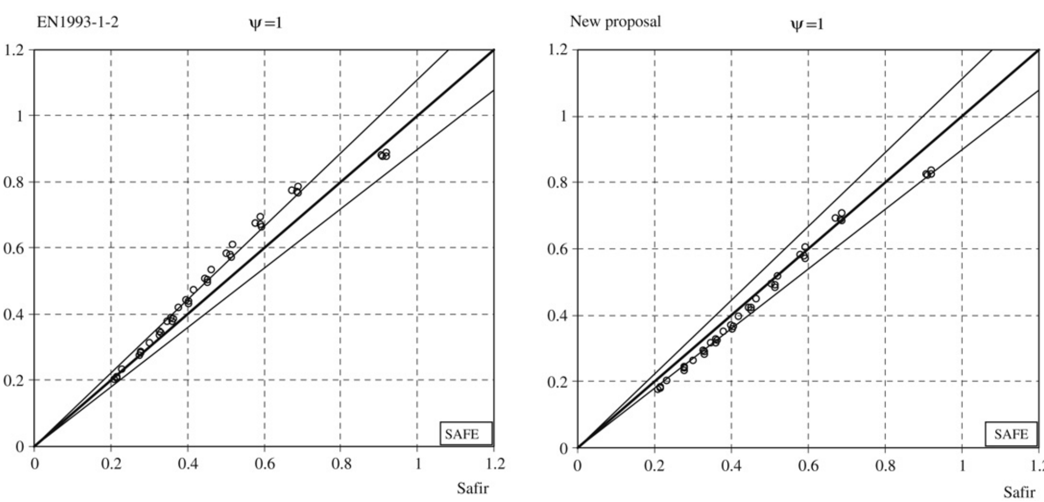


Fig. 21. Improvement for the case of $\psi = 1$ in IPE 500 beams in stainless steel 1.4301.

load and a uniformly distributed load, showing the influence of the loading type.

Figs. 13 and 14 presents the results obtained for the equivalent cross section HEA 500 and IPE 500 of the stainless steel grade 1.4301, illustrating the influence of the cross section slenderness.

Figs. 15–17 show the comparisons made for the equivalent cross section IPE 220 of the stainless steel grades 1.4401, 1.4571, 1.4462 and 1.4003, showing the influence of the stainless steel grade.

Finally Figs. 18 and 19 present the results obtained for the equivalent cross section IPE 500 of the stainless steel grade 1.4003, illustrating the influence of the cross section slenderness in the ferritic stainless steel.

To highlight the better accuracy and safety of the new proposal when considering the loading type, the effect of the factor f (Eq. (9)) can be seen in Figs. 20 and 21, which compare the numerical results with the two approaches (results from EN 1993-1-2 and from the new proposal), showing differences about 10%. From these figures, that compares the ratio $M/M_{fi,\theta,Rd}$, the same ratio plotted in the vertical axis of Figs. 4–7 and 12–19, it is clear that the new proposal is safer and more accurate than the current Eurocode 3 approach.

6. Plateau length for lateral–torsional buckling in stainless steel elements

Part 1.4 of Eurocode 3 states that for $\overline{\lambda}_{LT} \leq 0.4$ or $M_{Ed}/M_{cr} \leq 0.16$ no lateral–torsional buckling check is required at room temperature. In fire design, according to part 1.2 of Eurocode 3, it is always necessary to take into account the influence of the lateral–torsional buckling. However, if the influence of non-uniform bending is considered, lateral–torsional buckling can be neglected for higher slenderness limit values. This assumption is illustrated in Fig. 12. There is a plateau and its length depends on the shape of the bending diagrams.

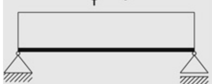
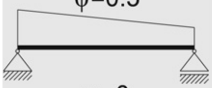
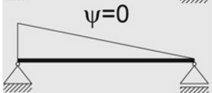
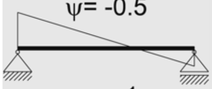
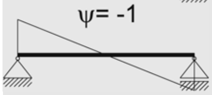
For the proposal made in this paper, in Section 4, lateral–torsional buckling can be neglected for slenderness values less than the values given in Table 5.

7. Conclusions

In this paper a new proposal for the lateral–torsional buckling of stainless steel elements was presented, considering the influence of the loading type.

Figs. 12–19 show that the proposal made by the authors [14], for the lateral–torsional buckling of carbon steel beams at high temperatures, that considers the influence of the loading type,

Table 5
Plateau length for equivalent welded stainless steel elements, at high temperatures, using the new proposal

β	0.85	1.00	0.85	0.85	1.00	1.20	1.00	1.00
f_y (N/mm^2)	460	460	220	210	250	250	220	210
Bending diagrams	$\bar{\lambda}_{LT,\theta}$							
$\psi=1$ 	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
$\psi=0.5$ 	0.189	0.162	0.133	0.130	0.121	0.118	0.113	0.111
$\psi=0$ 	0.367	0.320	0.267	0.262	0.245	0.238	0.231	0.226
$\psi=-0.5$ 	0.472	0.419	0.355	0.348	0.327	0.326	0.309	0.303
$\psi=-1$ 	0.507	0.452	0.385	0.378	0.355	0.351	0.336	0.330
	0.187	0.160	0.131	0.128	0.119	0.033	0.112	0.109
	0.077	0.065	0.053	0.052	0.048	0.117	0.045	0.044

gives results that are in good agreement with the numerical results obtained with the program SAFIR for stainless steel beams in case of fire, provided that a new imperfection factor is used.

It was concluded that the slenderness of the cross-section, assessed using the height/width (h/b) relation, should be taken into account as it is already proposed in Eurocode 3 for carbon steel elements at room temperature.

This paper has also shown that different severity factors should be used for the ferritic stainless steel grade 1.4003.

It is evident that for these ferritic grades, the use of Eq. (15) for the imperfection factor will give a more economic design procedure.

Finally, slenderness limit values (see Table 5) were presented that allow us to ignore lateral–torsional buckling, as a function of the bending diagrams, according to the proposal made in this paper. For non-dimensional slenderness less than the ones given in that table, lateral–torsional buckling may be ignored and only cross sectional checks apply.

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