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# Axially Loaded Stainless Steel Columns in Case of Fire

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## Abstract

The simple model of Eurocode 3, for the fire resistance evaluation of stainless steel members, are based on the procedures used for carbon steel structural elements. However, due to the existing differences in the constitutive laws of these two materials, it is expected that it would not be possible to use, in both materials, the same formulae for the member stability calculation, as proposed in Eurocode 3. This paper aims at increasing the knowledge on the behaviour of stainless steel axially loaded columns at elevated temperatures. For this purpose, a geometrical and material non linear computer code has been used to determine the buckling load of these elements. The Eurocode formulae are evaluated and a new proposal, that ensures accurate and conservative results when compared with the numerical simulations, is presented.

## NOTATIONS

### Roman

| | |
|---|---|
| $A$ | area |
| $b$ | width of a cross-section |
| $E$ | Young's modulus |
| $f_y$ | Yield strength |
| $h$ | depth of a cross-section |
| $k_{y,\theta}$ | reduction factor for the yield strength at temperature |
| $k_{E,\theta}$ | reduction factor for the slope of the linear elastic range at temperature |
| $L$ | length of the element |
| $N$ | axial force |
| $N_{b,fi,t,Rd}$ | buckling resistant axial force in case of fire |
| $N_{fi,Rd}$ | resistant axial force in case of fire |
| $N_{Safir}$ | ultimate axial force obtained in the SAFIR simulations |

### Greek

| | |
|---|---|
| $\alpha$ | Imperfection factor |
| $\beta$ | coefficient for determining |
| $\gamma_{M,fi}$ | Partial safety factor for the fire situation (usually ) |

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$\varepsilon$ coefficient for the cross-section classification
$\eta$ coefficient to determine
$\bar{\lambda}_i$ element slenderness value in axis
$\bar{\lambda}_{i,\theta}$ element slenderness value in axis at high temperatures
$\phi_{i,\theta}$ coefficient for determining the reduction factor
$\chi_{i,fi}$ reduction factor for the flexural buckling in axis in case of fire
$\chi_{\min,fi}$ minimum of the reduction factor $\chi_{i,fi}$

## 1. INTRODUCTION

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 intended to add value to 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 on the additional benefits of its use as a structural material, are some of the reasons that force designers to avoid its use. However, more accurate analyses point to a good performance of stainless steel when compared against conventional carbon steel in fire situation [1, 2, 3].

The most important advantage of stainless steels is their corrosion resistance, however, their aesthetic appearance, ease of maintenance, durability and the low life-cycle costs are also valuable characteristics. Engineers often disregard these advantages of stainless steel due to its high initial cost. Nevertheless, greater importance is being given to total life costing because of high maintenance, shut-down, demolition and parts replacement costs. Experience has shown that the benefits of a long life with low maintenance and repair requirements more than compensates for the higher initial purchase cost of stainless steel.

Part 1-4 of Eurocode 3 (EC3) "Supplementary rules for stainless steels" [4] gives design rules for stainless steel structural elements at room temperature, and only mentions the stainless steel structural elements 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, EC3 states that the structural elements made of these two materials must be checked for their fire resistance with the same formulae. Thus, based on the formulae in Part 1-4 of EC3, Uppfeldt [6] presented a design model for stainless steel hollow columns in case of fire based on both experimental and numerical tests. Regarding the study of axially loaded carbon steel columns, Franssen in 1996 [7, 8] proposed a procedure for the safety evaluation of columns subjected to high temperatures, later adopted by Part 1-2 of EC3 [5].

In this work the accuracy and safety of the currently prescribed design rules in EC3 for the evaluation of the resistance of stainless steel uniform columns at high temperatures [5] are evaluated. This evaluation is carried out by performing numerical simulations on Class 1 and Class 2 [4] stainless steel I-cross-sections. Comparisons between the numerical results and the buckling curves from EC3 for those elements are presented. Based on these comparisons, safer and more accurate proposals for the flexural buckling of stainless steel columns are presented.

The results obtained with EC3 are compared with the numerical results obtained with the program SAFIR [9, 10] using the methodology usually designated by GMNIA (geometrically and materially non-linear imperfect analysis). This program is a geometrical and material non-linear finite element code, specially developed at the University of Liege for the study of structures in case of fire. To model the behaviour of stainless steel structures, the program has been adapted according to the material properties defined in Part 1-4 and Part 1-2 of EC3 [4, 5].

This adaptation was made through the introduction of the stainless steel mechanical properties in a one-dimensional constitutive model (used on the truss and beam finite elements). In this model, in order to take into consideration unloading effects, the rule of Masing [11] was implemented, similarly to the way it was introduced for the carbon steel model in SAFIR [12]. The main difference lies in the fact that the stainless steel does not have an initial linear phase in the stress-strain relationship (see Figure 1).

In cyclic loading with decreasing stress levels, the implemented algorithm does not cope with the rule of Masing. However, this case is not common on buildings structures submitted to heating.

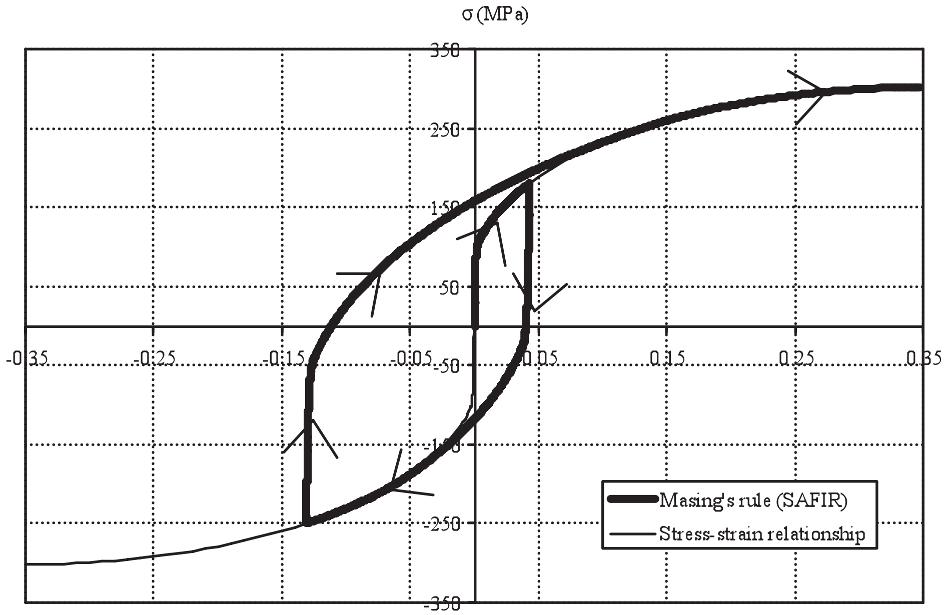



Figure 1. Masing's rule in stainless steel grade 1.4301 at 600 ºC.

Finally, the consideration of the residual stresses is made through the introduction of residual strains [13, 14].

With the results obtained from the numerical study, it is shown that the design formulae for stainless steel columns proposed in EC3 do not provide good approximations to the real behaviour. EC3 as shown to be sometimes uneconomical, and more important and critical, in other cases unsafe.

The study presented in this paper resulted in the proposal of safer buckling curves. It is shown that these buckling curves at high temperatures [5] should vary with the buckling axis, as it is already prescribed at room temperature, in Part 1-4 of EC3 [4].

Stainless steels have non-linear stress-strain relationship with a low proportional stress and an extensive hardening phase [15, 16]. There is no well defined yield strength and the 0.2 % proof strength, $f_y = f_{0.2p}$, is usually considered for room temperature design. In a fire situation, higher strains than at room temperature are acceptable, and Part 1-2 of EC3 suggests the use of the stress at 2 % total strain $f_{y,\theta} = f_{2,\theta}$ as the yield strength to be used in simple design equations at elevated temperature $\theta$ [17], for Class 1, 2 and 3 cross-sections. Other proposals can be found in the literature, for the evaluation of stainless steel members in case of fire, that adopt the proportional limit strain of 0.2 % [18, 6]. As the use of higher deformations (2 % total strain) is recommended in fire design from EC3, this assumption is considered in the proposals presented in this work, preserving the philosophy of the Eurocode.

## 2. PRESCRIBED DESIGN RULES

In this section the design rules for stainless steels columns, prescribed in EC3, will be presented. It is also shown that they should be improved, in order to better approximate the numerical results

### 2.1 Classification of the sections

The classification of stainless steel cross-sections at room temperature is made, according to EN 1993-1-4 [4], using a factor $\varepsilon$ that is a function of the yield strength and of the Young's modulus.

$$\varepsilon = \left( \frac{235}{f_y} \frac{E}{210000} \right)^{0.5} \quad \text{with } f_y \text{ and } E \text{ in MPa} \tag{1a}$$

The same equation could be given in EN 1993-1-1, but as the Young's modulus at 20 ºC is equal to 210 GPa [19] for all carbon steel types, the formula is written as presented in Table 1.

Table 1. Factor used for the cross-section classification at room temperature.

| | |
|---|---|
| In Part 1-1 of EC3 for carbon steel sections | $\varepsilon = \left(\frac{235}{f_y}\right)^{0.5}$ |
| In Part 1-4 of EC3 for stainless steel sections | $\varepsilon = \left(\frac{235}{f_y}\frac{E}{210000}\right)^{0.5}$ |

If the variation of the yield strength and of the modulus of elasticity with increasing temperatures are directly introduced in eqn (1a), the factor $\varepsilon$ for the cross-sectional classification at elevated temperature can be calculated according to eqn (1b).

$$\varepsilon = \left(\frac{235}{f_{y,\theta}}\frac{E_\theta}{210000}\right)^{0.5} = \left(\frac{235}{f_y}\frac{E}{210000}\right)^{0.5}\left(\frac{k_{E,\theta}}{k_{y,\theta}}\right)^{0.5} \quad \text{with } f_y \text{ and } E \text{ in MPa} \tag{1b}$$

As can be seen in Figure 2 the factor $\varepsilon$, for carbon steel, reduces for high temperatures, meaning that the classification made at 20 ºC is unsafe for high temperatures. Due to this, Part 1-2 of EC3 [5] recommends the use of a reduced factor for the fire design of carbon steel given by

$$\varepsilon = \left(\frac{k_{E,\theta}}{k_{y,\theta}}\right)^{0.5}\left(\frac{235}{f_y}\right)^{0.5} \approx 0.85\left(\frac{235}{f_y}\right)^{0.5} \tag{2}$$

Stainless steel exhibits a different behaviour from carbon steel, but with a factor normally bigger than the one given by eqn (2), as shown in Figure 2. Therefore it can be concluded that using this

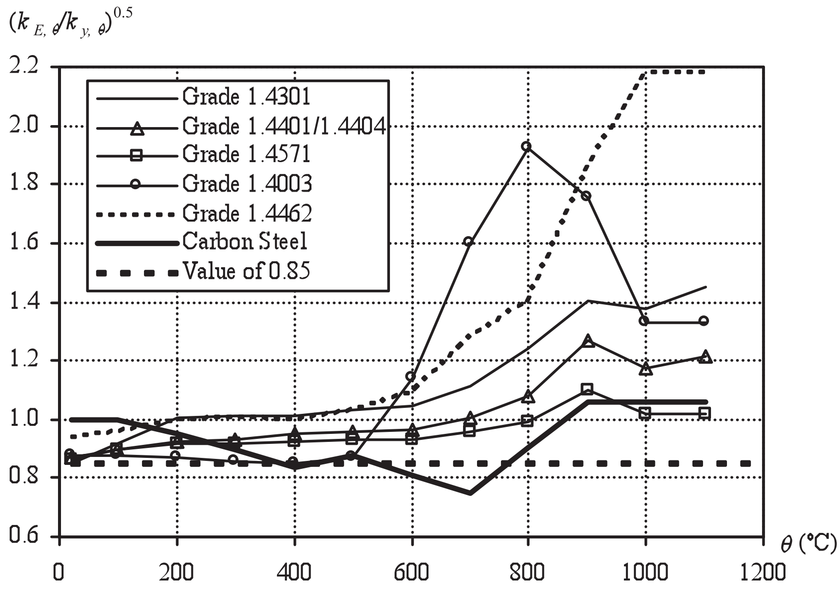



Figure 2. Reduction of the factor to determine the cross-section classes, at high temperatures.

equation to determine the classification of stainless steel cross-section in case of fire is safer than using it for carbon steel.

In Figure 2 it can be observed that the steel grades 1.4003 and 1.4462 have higher reduction factor ratios than other steel grades. This fact is due to the bigger yield strengths reductions, of these two stainless steel grades, at high temperatures [5, 20].

## 2.2 Design equation for stability

The calculation of the compression resistance of columns made of Class 1 and 2 sections, subjected to high temperatures (Figure 3) recommended by Part 1-2 of EC3 was proposed by Franssen [8] and was based on extensive numerical simulations calibrated on the base of experimental tests on carbon steel columns.

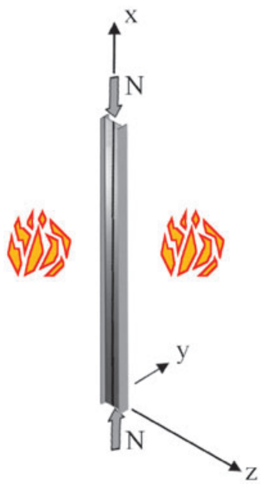



Figure 3. Element subjected to axial compression in case of fire.

For stainless steel structural elements subjected to high temperatures, Part 1-2 of EC3 states that the flexural buckling resistance, for Class 1, 2 and 3 sections, is given by

$$N_{b,fi,t,Rd} = \chi_{\min,fi} A k_{y,\theta} \, f_y / \gamma_{M,fi} \tag{3}$$

where the reduction factor $\chi_{\min,fi}$ is the minimum of the $\chi_{y,fi}$ and $\chi_{z,fi}$, which are determined with the expression

$$\chi_{i,fi} = \frac{1}{\varphi_{i,\theta} + \sqrt{\varphi_{i,\theta}^2 - \bar{\lambda}_{i,\theta}^2}} \quad \text{with} \quad \chi_{i,fi} \le 1 \tag{4}$$

with

$$\varphi_{i,\theta} = \frac{1}{2}\left(1 + \alpha \bar{\lambda}_{i,\theta} + \bar{\lambda}_{i,\theta}^2\right) \tag{5}$$

In this expression, the imperfection factor $\alpha$ depends on the steel grade and is determined by

$$\alpha = 0.65\varepsilon \tag{6}$$

where $\varepsilon$ is given in Part 1-1 of EC3 [19] as

$$\varepsilon = \sqrt{235 / f_y} \tag{7}$$

The imperfection factor is then given by

$$\alpha = 0.65\sqrt{235 / f_y} \tag{8}$$

The normalized slenderness for buckling at high temperatures is obtained with

$$\bar{\lambda}_\theta = \bar{\lambda} \left[ \frac{k_{y,\theta}}{k_{E,\theta}} \right]^{0.5} \tag{9}$$

The flexural buckling safety evaluation, for profiles with cross-section Class 4, follows the same procedure, considering the effective section area $A_{eff}$ instead of the gross cross-section $A$ and assuming $f_{y,\theta} = f_{0.2p,\theta}$.

Finally, Part 1-2 of EC3 recommends the use of the value 1.0 for the partial safety factor in fire situation $\gamma_{M,fi}$.

### 2.3 Comparison against numerical results

Figure 4 shows, as an example, the buckling curve from the EC3 (denoted "EN 1993-1-2") compared with the numerical values of a HE200A profile of stainless steel grade 1.4301 subjected to elevated temperatures, for the strong and for the weak axis. In the graphs $N_{fi,Rd} = Ak_{y,\theta} f_y / \gamma_{M,fi}$. It can be observed that the recommended buckling curve is not on the safe side compared to the numerical results.

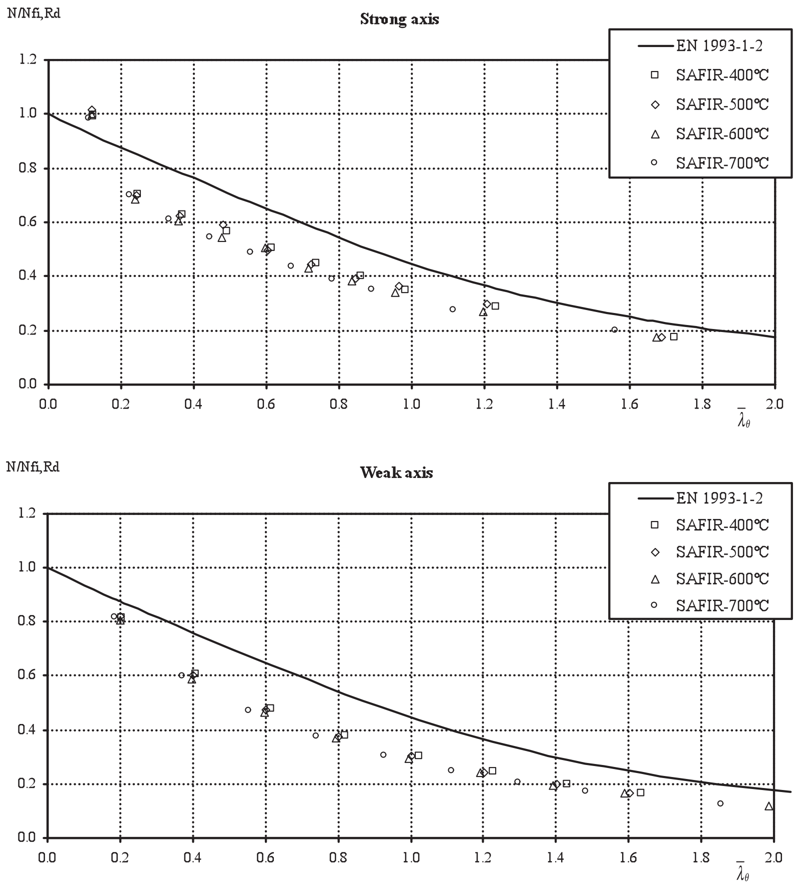



Figure 4. Comparison between the curve from EC3 and the numerical results: buckling about the strong axis and the weak axis, at high temperatures, for HE200A in stainless steel grade 1.4301.

## 3. CASE STUDY

For the parametric study presented in this paper, simply supported column were used. The columns were subjected to axial compression with the possibility of occurring flexural buckling about the strong axis or about the weak axis.

In the numerical simulations, a geometric imperfection given by the following expression was considered [21]:

$$y(x) = \frac{L}{1000} \sin\left(\frac{\pi x}{L}\right) \tag{10}$$

Residual stresses distribution, having the maximum value of the yield strength, similar to the one used for the welded carbon steel sections [22, 23, 24], were introduced as shown in Figure 5.

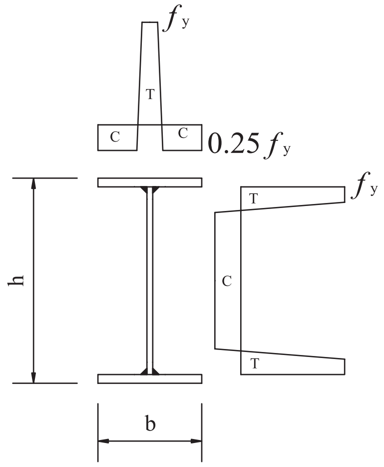



Figure 5. Residual stresses in I-welded sections (C – compression; T – tension).

Welded cross-sections equivalent to HE200A, HE280B and HE200B were used. The types of stainless steel grade used were: 1.4301, 1.4401 or 1.4404, 1.4571, 1.4003 and 1.4462 [4, 5]. Table 2 shows all the cases that have been studied, chosen so that the cross-section were of Class 1 or 2. For the simulations in case of fire, the temperatures of 400, 500, 600 and 700 ºC, uniform in the all cross-section, were used. An average of 10 lengths were analysed for each case presented in Table 2.

Table 2. Cases studied.

| | **400, 500, 600 and 700 ºC** | |
|---|---|---|
| | **Strong axis** | **Weak axis** |
| HE200A | | |
| 1.4301 | ✓ | ✓ |
| 1.4401 | ✓ | ✓ |
| 1.4404 | ✓ | ✓ |
| 1.4571 | ✓ | ✓ |
| HE280B | | |
| 1.4301 | ✓ | ✓ |
| 1.4401 | ✓ | ✓ |
| 1.4404 | ✓ | ✓ |
| 1.4571 | ✓ | ✓ |
| 1.4003 | ✓ | ✓ |
| HE200B | | |
| 1.4003 | ✓ | ✓ |
| 1.4462 | ✓ | ✓ |

## 4. DESIGN PROPOSAL

Based on the comparison between the numerical results and the design formulae given in Part 1-2 of EC3 [5] for stainless steel columns in case of fire (presented in section 2), an improved proposal is made. It will be shown that this new proposal is safer than the formulae from EC3.

The stainless steel mechanical properties at high temperatures vary significantly with the steel grade. This fact could lead to the use of different design formulae for each grade. In the proposal presented here, those different behaviours were taken into account, but with the minimum introduction of complexity, so that only one set of design formulae was developed for all the stainless steel grades, with the exception of the stainless steel 1.4462, which needs a separate proposal due to its high value of the yield strength.

### 4.1 Equations of the new proposal

Each stainless steel grade has a different structural behaviour when subjected to high temperatures [5]. Their yield strength reductions at high temperatures, $k_{y,\theta}$, are rather different from each other. As it can be seen in eqn (11), according to EC3, the normalized slenderness at room temperature, $\bar{\lambda}$, is multiplied by the relation $\left(k_{y,\theta}/k_{E,\theta}\right)^{0.5}$ in order to obtain the slenderness in case of fire.

$$\bar{\lambda}_\theta = \bar{\lambda}\left[\frac{k_{y,\theta}}{k_{E,\theta}}\right]^{0.5} \tag{11}$$

Figure 6 presents the variation of the square root, in eqn (11), as a function of the temperature (in fact, the inverse of the value presented on Figure 2). This graph suggests that a different buckling curve should be provided for each stainless steel grade.

From Figure 6 it can be observed that, from 500 ºC to 700 ºC, there is a great decrease of the relation $\left(k_{y,\theta}/k_{E,\theta}\right)^{0.5}$ for the 1.4003 stainless steel, which does not occur with the other stainless steel grades.

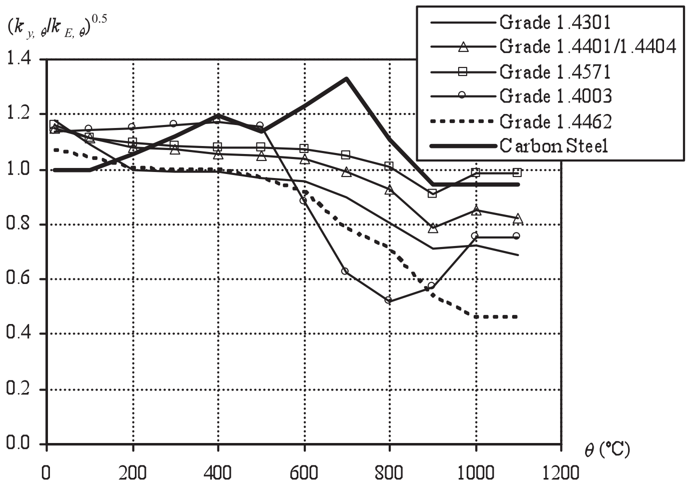



Figure 6. Slenderness variation at high temperatures.

In the proposal presented in this paper, the necessity of different buckling curves, for different stainless steel grades, will be taken into consideration.

In this new proposal, the buckling resistance value of a compression element is calculated as in Part 1-2 of EC3 with

$$N_{b,fi,t,Rd} = \chi_{\min,fi} A k_{y,\theta} \, f_y / \gamma_{M,fi} \tag{12}$$

where the reduction factor $\chi_{\min,fi}$ is the minimum of the values $\chi_{y,fi}$ and $\chi_{z,fi}$.

This proposal was developed in order to minimize the changes in the formulae of EC3. It was adopted a factor $\beta$ similar to the one used in Part 1-1 of EC3 for the design of beams in carbon steel with LTB [19], according to the following formulae

$$\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^2 - \beta\bar{\lambda}_\theta^2}} \tag{13}$$

with

$$\varphi_\theta = \frac{1}{2}\left[1 + \alpha\bar{\lambda}_\theta + \beta\bar{\lambda}_\theta^2\right] \tag{14}$$

where the factor $\beta$, based on a statistical evaluation (see section 4.2), depends on the buckling axis as presented in Table 3.

Table 3. Coefficient for determining the reduction factor.

| | $\beta$ |
|---|---|
| Strong axis | 1.0 |
| Weak axis | 1.5 |

A new imperfection factor $\alpha$, function of a severity factor $\eta$ is used, based on eqn (6).

$$\alpha = \eta\varepsilon \tag{15}$$

where $\varepsilon$ is given by eqn (1b) in order to consider the different behaviours at high temperature of the different stainless steel grades shown in Figure 6. The imperfection factor can thus be written as a function of the temperature as

$$\alpha = \eta\sqrt{\frac{235}{f_y}\frac{E}{210000}}\sqrt{\frac{k_{E,\theta}}{k_{y,\theta}}} \tag{16}$$

The proposal for the values of the factor $\eta$, to be used with this equation, was also based in the statistical evaluation and is given in Table 4. It was necessary to account for the higher yield strength value of the duplex grade (1.4462), using a lower severity factor has shown in the table.

Table 4. Severity factor for the flexural buckling of stainless steel elements in case of fire.

| | **1.4301, 1.4401, 1.4404, 1.4571, 1.4003** | **1.4462** |
|---|---|---|
| $\eta$ | 1.3 | 0.9 |

## 4.2 Accuracy of the proposal

In this section, the validation of this new proposal for the fire design of stainless steel columns at elevated temperatures is presented.

### Numerical validation

Results for stainless steel columns subjected to flexural buckling about the strong axis and about the weak axis are shown in Figure 7. Only the results for a HE200A of the stainless steel grade 1.4301 are here presented.

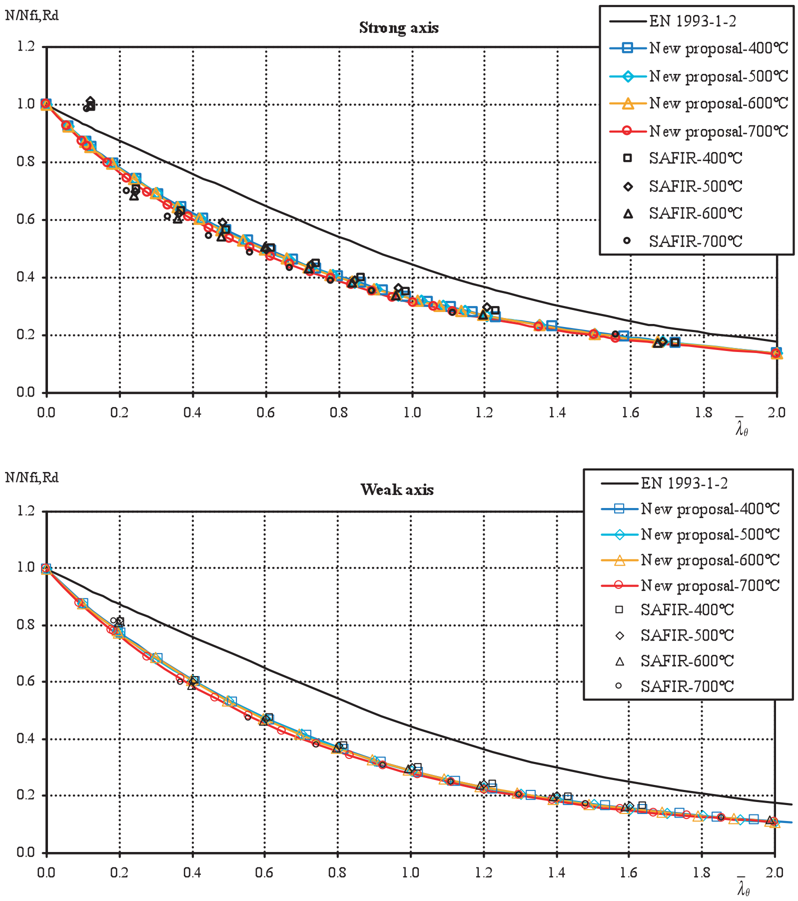


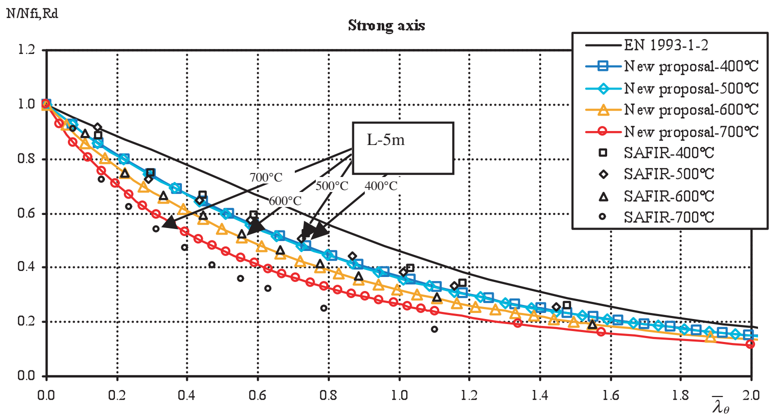

Figure 7. Column with cross-section HE200A in stainless steel grade 1.4301, at high temperatures.

A comparison is made between the curve obtained through the formulae from Part 1-2 of EC3 described in section 2 of this paper (denoted "EN 1993-1-2" in the graphs), the curve based on the proposal made here (denoted "New proposal"), and the numerical results determined with SAFIR.

Figure 7 shows that the new proposal improves the accuracy of the buckling curves obtained with the formulae prescribed in Part 1-2 of EC3, which are not safe when compared with the numerical results.

Although the graphs may suggest that the buckling curve does not depend on the temperature, introducing this dependence in the model made possible to use the same equations for the other stainless steel grades without having to change the imperfection factor or any other coefficient.

More significant, the observed relation $\left(k_{y,\theta}/k_{E,\theta}\right)^{0.5}$ for the ferritic stainless steel grade 1.4003 introduces, in the temperature range of 500 ºC to 700 ºC, a high variation of the normalised slenderness at high temperatures, which lead to the results shown in Figure 8. As it can be observed, the slenderness values for 700 ºC and 600 ºC are quite different from the corresponding values for 400 ºC and 500 ºC. With the presented proposal this behaviour can be more accurately predicted. It can be observed that the results obtained with the new proposal for stainless steel grade 1.4003 columns at 700 ºC are not on the safe side. This was accepted in order to keep the complexity of the proposal to a level that is acceptable for simple design equations.



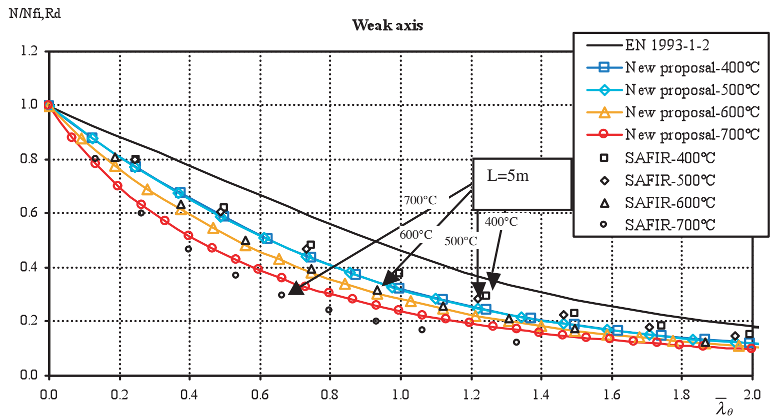




Figure 8. Column with cross-section HE200B in stainless steel grade 1.4003, at high temperatures.

## Statistical evaluation

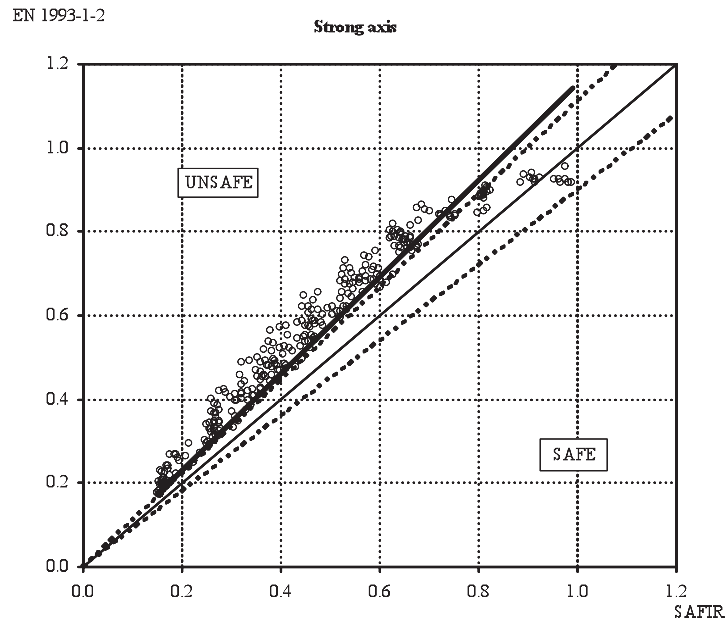
For each buckling axis, the ratio between the analytical value of the ultimate design axial force $N_{ult,i}^{Analytical}$ and the corresponding SAFIR axial force $N_{ult,i}^{SAFIR}$, has been evaluated. This ratio is also the ratio between the analytical and the SAFIR reduction factor for the flexural buckling, as follows:

$$x_i = \frac{N_{ult,i}^{Analytical}}{N_{ult,i}^{SAFIR}} = \frac{\chi_i^{Analytical}}{\chi_i^{SAFIR}} \tag{17}$$

Figures 9 and 10 plot the ratios $x_i$ for 308 and 294 results, respectively for the strong and weak axis, obtained with EN 1993-1-2 for welded HE200A, HE280B and HE200B sections in stainless steel grades 1.4301, 1.4401, 1.4571, 1.4003 and 1.4462. The average value and the standard deviation are, respectively, $\mu$ = 1.169 and $s$ = 0.250 for the strong axis, and $\mu$ = 1.345 and $s$ = 0.194 for the weak axis (see Table 5).

Table 5. Statistical results of columns at elevated temperatures.

| | | **EN 1993-1-2** | **New proposal** |
|---|---|---|---|
| Strong axis | Average value | 1.169 | 0.978 |
| | Standard deviation | 0.250 | 0.049 |
| Weak axis | Average value | 1.345 | 0.956 |
| | Standard deviation | 0.194 | 0.076 |



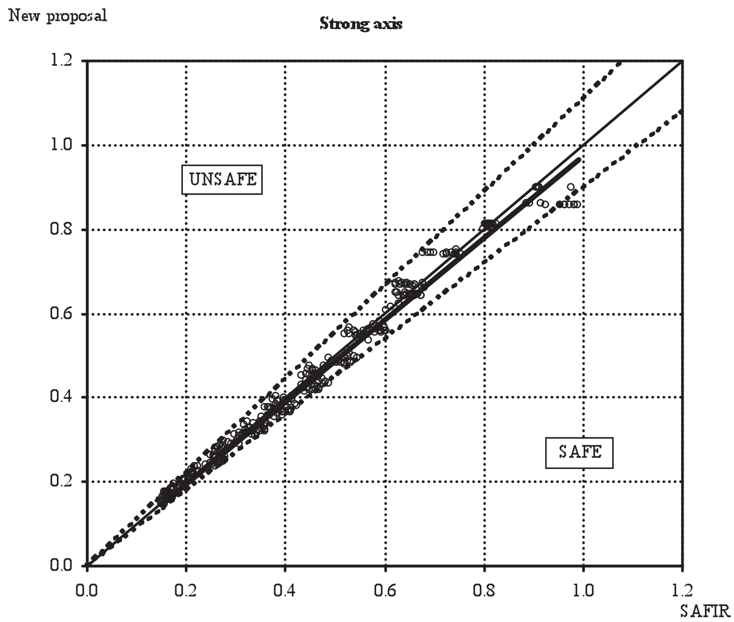




Figure 9. Comparison between EN 1993-1-2, the new proposal and numerical results for the strong axis, at high temperatures.

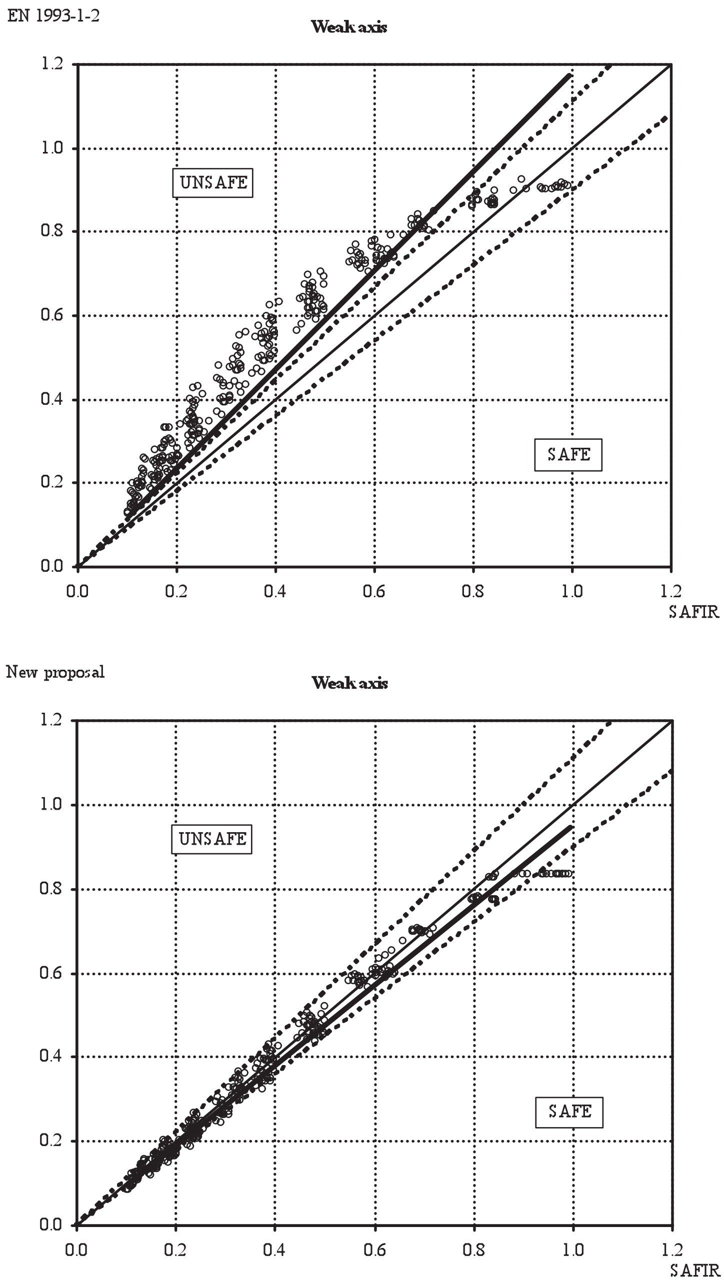



Figure 10. Comparison between EN 1993-1-2, the new proposal and numerical results for the weak axis, at high temperatures.

If the new proposal is used in those numerical results, the average value and the standard deviation take the values $\mu = 0.978$ and $s = 0.049$ for the strong axis, and $\mu = 0.956$ and $s = 0.076$ for the weak axis, showing that the new proposal is, in average, safer than the EC3, with a smaller value of the standard deviation.

Figures 11 and 13 show the ratio $x_i$ plotted as a function of the relative slenderness, when EN 1993-1-2 is used, with all the numerical results for the strong and weak axis. The maximum unsafe error is 51.4 % and 87.6 % for the strong and for the weak axis, respectively. The average of the unsafe errors is 22.0 % and 36.2 % for the strong and for the weak axis, respectively.

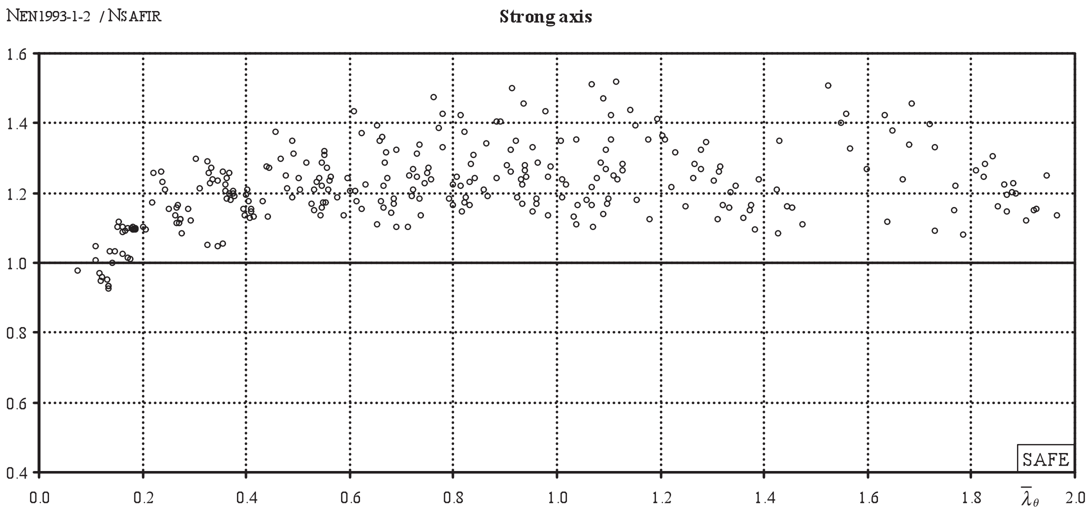



Figure 11. Ratio between analytical and SAFIR results, for EN 1993-1-2 in the strong axis, at high temperatures.

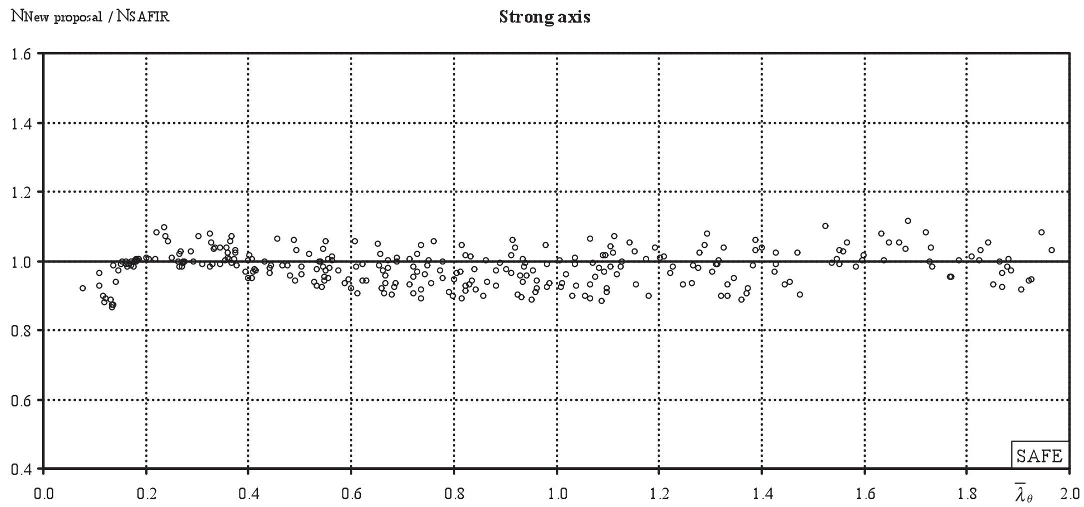



Figure 12. Ratio between analytical and SAFIR results, for the new proposal in the strong axis, at high temperatures.

The new proposal gives the results shown in figures 12 and 14, with the maximum unsafe error of 11.2 % and 13.4 % for the strong and weak axis, respectively, and an average of the unsafe errors of 3.2 % and 3.9 % for the strong and weak axis, respectively. These last values are smaller than the corresponded values obtained with EN1993-1-2, suggesting again that the proposal is safer.

From these graphs it can be concluded that the new proposal gives a much safer approximation to the numerical results than the EC3.

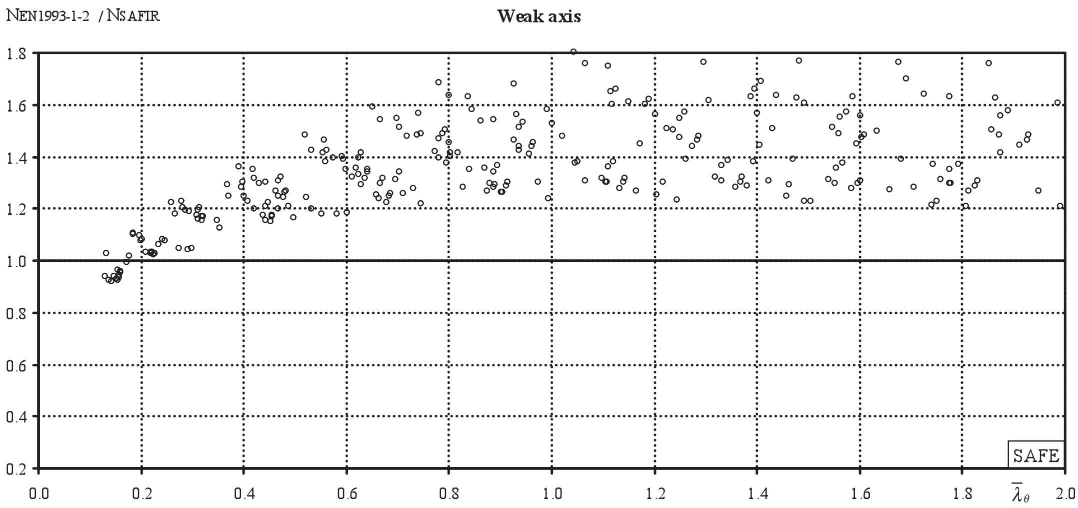



Figure 13. Ratio between analytical and SAFIR results, for EN 1993-1-2 in the weak axis, at high temperatures.

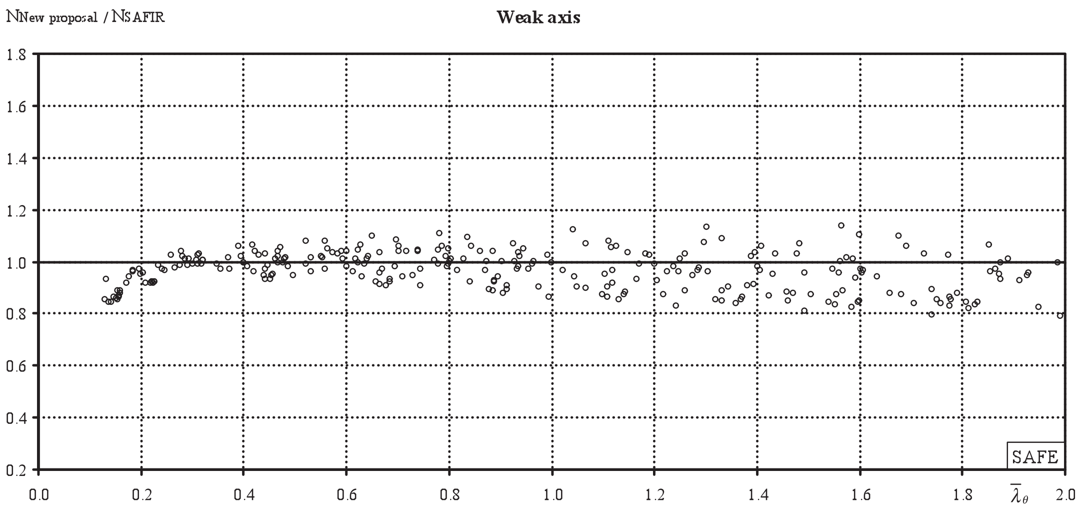



Figure 14. Ratio between analytical and SAFIR results, for the new proposal in the weak axis, at high temperatures.

## 5. CONCLUSIONS

This paper presents a numerical study on the behaviour of axially loaded stainless steel columns in case of fire, using GMNIA (geometrically and materially non-linear imperfect analysis). Buckling about the strong and weak axis, different stainless steel grades, different elevated temperatures and different slenderness values were considered.

It was shown that the design formulae originally developed for carbon steel elements and included in Part 1-2 of EC3 [5] are unsafe for the flexural buckling of welded I-stainless steel profiles at high temperatures, about the strong as well as about the weak axis. It has also been shown that the buckling curves at high temperatures should vary with the buckling axis, as is the case in Part 1-4 of EC3 for room temperature design.

A new proposal has been presented, that leads to results much closer to the results of numerical calculations. Due to the fact that the stress-strain relationships at elevated temperatures depend on the steel grade, it was necessary to account for this influence, mainly for the duplex grade (1.4462).

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