Class 4 Stainless Steel I Beams Subjected to Fire

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Abstract

Structural elements composed of Class 4 sections are common in Stainless Steel buildings structures. These thin walled profiles are more susceptible to the occurrence of local buckling. Additionally, in beams the lateral-torsional buckling is also a common failure mode. These instability phenomena are intensified at high temperatures. This work has the main objective of presenting a numerical study on the fire behaviour of beams with Class 4 stainless steel sections when subjected to pure bending and high temperatures. The influence of several parameters, as geometrical imperfections and residual stresses, on the ultimate load will be evaluated and comparisons between the numerical results and the Eurocode 3 rules will also be made.

Keywords

Fire, Beams, Class 4 sections, Stainless steel

1   Introduction

The stainless steel has countless desirable characteristics for a structural material [1, 2 and, 3]. Although initially more expensive than conventional carbon steel, stainless steel structures can be competitive because of its smaller need for thermal protection material, and lower life-cycle cost, thus contributing to a more sustainable construction.

Stainless steel grades can be classified according to their metallurgical structure in five groups: the austenitic, ferritic, martensitic, austenitic-ferritic and precipitation-hardening groups [1]. Austenitic stainless steels provide a good combination of corrosion resistance, forming and fabrication properties. The ferritic stainless steels have lower percentage of Niquel which can reduce its price still providing relatively good corrosion resistance and strength. Duplex (austenitic-ferritic) 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 [4] and austenitic-ferritic steels [5] for structural purposes due to the mentioned above advantages.

Regarding fire structural resistance, simplified design rules provided by codes of practice, are of the utmost importance to designers that do not always have access to applications dealing with advanced calculation methods.

The Part 1.4 of Eurocode 3 (EC3) EN 1993-1-4 [6] provides design rules for stainless steel structural elements at normal temperature, and for their fire resistance it refers to the Part 1.2 of the same Eurocode, EN 1993-1-2 [7]. In this Part 1.2 of EC3 it is stated that structural elements made of stainless steel must be checked for its fire resistance using the same design formulae developed for carbon steel. However, as it is also presented in the annex C of EN 1993-1-2, these two materials have different constitutive laws in case of fire (they have also at normal temperature different stress-strain relationships).

The phenomenon of lateral torsional buckling (LTB) in carbon steel beams in case of fire was addressed in previous research works [8 and 9], which led to the proposal of an expression that was adopted in Part 1-2 of EC3. This proposal was developed for the case of simply supported Class 1 and 2 (stocky profiles [10]) beams with fork supports and uniform bending diagrams. Aditionally, a study on the LTB of stainless steel beams in case of fire was also developed on Class 1 and 2 cross-sections [11]. However, the use of stainless steel members with thin-walled cross-sections in buildings is common due to its lightness and long span capacity.

In fact, the knowledge of the performance of Class 4 steel elements (slender sections more susceptible to the occurrence of local buckling) in case of fire is still limited. It was introduced in the French National Annex of the EN1993-1-2 new design proposals for Class 4 carbon steel structural elements in case of fire, acknowledging the conservative nature of the prescribed simplified rules in EN 1993-1-2. Additionally the manual of EuroInox [1] also proposes the use of different formulae for the fire design of stainless steel elements. Thus is of great interest to better understand the behaviour of these thin-walled stainless steel beams with LTB in case of fire.

The structural steel elements with thin walled sections, subjected to simple bending, are characterized by being able to have the possibility of occurrence of both failure modes local buckling and LTB. These instability phenomena and its influence on the ultimate strength are of upmost importance to characterize the behaviour of these members. Thus, the program CUFSM was used for the purpose of obtaining the local and global instability modes of the studied cross

sections. The obtained buckling modes were used to define the geometrical imperfections to be used on the finite element models. The program CUFSM performs elastic buckling analysis of thin-walled elements using the finite strip method (FSM) [12].

The main objective of this work was to perform a numerical analysis using the software SAFIR [13], a geometrical and material non linear finite element code especially developed to model the behaviour of structures in case of fire, on thin-walled stainless steel structural elements subjected to bending and in case of fire. These numerical results are compared against the buckling curves from the different design approaches mentioned above for those elements.

2 Material behaviour

Stainless steels are known for their non-linear stress-strain relationships with a low proportional stress and an extensive hardening phase [8 and 9]. There is not a well defined yield strength, being usually considered for design at room temperature the 0.2% proof strength, $f_y = f_{0.2\,proof}$. 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% [10] total strain as the yield stress at elevated temperature θ, $f_{y,\theta} = f_{2,\theta}$, for Class 1, 2 and 3 cross-sections and $f_{y,\theta} = f_{0.2\,proof,\theta}$, for Class 4 which is the proof strength at 0.2% plastic strain, 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 EC3 [7], is shown in Figure 1 and Figure 2 where $k_{0.2\,proof,\theta} = f_{0.2\,proof,\theta} / f_y$ and $k_{E,\theta} = E_\theta / E$.

From Figure 1 it can be observed that, according the EC3 [7], 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 EC3) with temperature is different from the other stainless steel grades. The reduction of the 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 paper. 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 modelling of stainless steel was made by a non-elastic plane stress condition, based on the von Mises surface and isotropic hardening [14]. The constitutive law of stainless steel has no elastic domain.

The implemented shell finite element was programmed to be used with large displacements in the context of plane stress condition. This finite element was first introduced for elastic materials and then for bi-dimensional elasto-plastic constitutive law [15].

The same formulation used in the carbon steel [15] was used for the stainless steel, but, due to the impossibility of inverting the stress-strain relationship (see Table 1 and Figure 3) for the first branch of the curve ($\varepsilon < \varepsilon_{c,\theta}$), an approximation had to be used [14].

The stainless steel stress-strain relationship introduced in the program fits with the one prescribed in part 1.2 of EC3 [7] for uniaxial relationship described in Table 1 and in Figure 3.

The constitutive law given by the function $\sigma = \sigma(\varepsilon)$ can be found in Table 1. The implemented hardening rule, $\tau = \tau(k)$, was obtained using $\varepsilon = k + \dfrac{\sigma}{E}$ and making $\tau = \sigma$ (see Figure 3).

For the second branch ($\varepsilon_{c,\theta} \leq \varepsilon < \varepsilon_{u,\theta}$) it was possible to use this procedure to achieve directly the hardening rule, but for $\varepsilon < \varepsilon_{c,\theta}$ the equation $\sigma = E \cdot \varepsilon /(1 + a \cdot \varepsilon^b)$ can not be inverted. Therefore, an equation, approximating very closely the hardening function for the first branch of the stainless steel constitutive law, was developed [14].

3 Lateral-torsional buckling fire design recomendations

In this section it is presented the fire design procedures for Class 4 steel beams analysed in this paper. The approaches prescribed in EN 1993-1-2, proposed in the French National Annex of this same part of EC3 and in the EuroInox design manual are here described.

3.1 Eurocode 3 formulae

For stainless steel beams subjected to elevated temperatures, Part 1.4 of EC3 [6] refers that the same formulation prescribed for carbon steel elements must be used. According to the EN 1993-1-2, the LTB resistant moment for Class 4 cross-sections, is

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{eff,y} k_{0.2\,proof,\theta} f_y \frac{1}{\gamma_{M,fi}}$$ (1)

where $W_{eff,y}$ is the effective section modulus of the cross-section [10] and $\chi_{LT,fi}$ is given by

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

with

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

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

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

The non-dimensional slenderness for LTB at high temperatures is given by

$$\bar{\lambda}_{LT,\theta} = \bar{\lambda}_{LT}\left[\frac{k_{0.2\,proof,\theta}}{k_{E,\theta}}\right]^{0.5}$$ (5)

where $\bar{\lambda}_{LT}$ is the non-dimensional slenderness at normal temperature.

3.2 Eurocode 3 French national annex formulae

The French national annex of EN 1993-1-2 proposes the use of the EN 1993-1-2 formulae with the following expression:

$$\phi_{LT,\theta} = \frac{1}{2}\left[1 + \alpha\left(\bar{\lambda}_{LT,\theta} - 0.2\right) + \left(\bar{\lambda}_{LT,\theta}\right)^2\right]$$ (6)

This equation enables the appearance of a plateau until slenderness of 0.2 on the buckling curve as it will be shown on the parametric study.

And the imperfection factor is obtained from Part 1.1 of EC3 using the buckling curve for hot rolled sections, which on the studied cases is equal to $\alpha = 0.34$.

3.3 EuroInox manual proposal

The EuroInox manual has also a different approach, proposing the use of a bigger plateau (0.4), which results from applying the equation:

$$\phi_{LT,\theta} = \frac{1}{2}\left[1 + \alpha\left(\bar{\lambda}_{LT,\theta} - 0.4\right) + \left(\bar{\lambda}_{LT,\theta}\right)^2\right]$$ (7)

And a higher imperfection factor, which on the studied cases is equal to $\alpha = 0.76$.

4 Buckling modes analyses

The prediction of geometrically nonlinear behaviour (of stability) of thin wall beams must be made using analysis methods covering the influence of each of the relevant instability phenomena.

This section presents the determined critical loads of elastic instability and corresponded modes, through numerical analysis carried out at the beams chosen for the parametric study presented in this paper with software CUFSM [12].

The chosen profiles were composed of welded I sections with height of 460 mm and width of 150 mm. One of the sections had 3 mm for the thickness of the web and 4 mm for the flanges (designated in this paper as 460x150x3x4) and the other had 4 mm on the web thickness and 7 mm on the flange thickness. Both sections are of Class 4 according to EN 1993-1-4 on the web in bending and on the compressed flange and they have different degrees of susceptibly to local buckling.

For a better understanding of the existing instability modes, it is presented in Figure 4 and Figure 5, an analysis of the instability modes that the profiles can suffer, when subjected to simple bending. The lower local buckling modes were obtained for the lengths of 340 mm on the section 460x150x3x4 and 270 mm on the section 460x150x4x7.

It can be observed that the beams with the section 460x150x4x7 have less susceptibly to local buckling as they have higher load ratio for the shorter lengths.

In Figure 6 and Figure 7 is presented the 2D and 3D deformed shape of the lower local buckling mode and of a length where LTB was the critical mode for the 460x150x3x4.

These buckling modes were used to define initial imperfection shapes on the finite element model as it will be shown in the next section.

5   Finite element model description

In this section it is described the numerical model adopted on the numerical parametric study performed with the finite element software SAFIR.

Simply supported beams with fork supports and submitted to uniform bending has shown in Figure 8 were analysed. As local buckling phenomena were likely to occur shell finite elements were used. The mesh size was chosen in order to capture all the possible failure modes.

The stainless steel grades considered in this study were the austenitic 1.4301, the ferritic 1.4003 and the austenitic-ferritic 1.4462, grades that have their material constitutive model at high temperatures described in EN 1993-1-2.

Due to the thin walls of these profiles and to the steel high thermal conductivity, the numerical tests were made with uniform temperatures in the cross section. The temperatures chosen were 350 ºC for being the critical temperature suggested to be used by EN 1993-1-2 when no calculation is performed, 500 ºC and 600 ºC deemed to be representative of the typical critical temperatures in steel structural elements.

Initial geometric imperfections and residual stresses were accounted for. In the following sections, it is presented a study of the influence of the existing initial imperfections and residual stresses on the ultimate load bearing capacity of the Class 4 stainless steel beams, in fire situation.

5.1   Influence of the geometric imperfections

To determine the shape of all the imperfections, the analysis performed with the program CUFSM considering the applied elastic stresses diagram corresponded to simple bending was used. The local and LTB instability modes shape obtained in CUFSM were used to define the geometrical imperfections. Thus in this study, the following situations were considered:

- without geometrical imperfections;

- with geometrical imperfections corresponded to the local buckling mode;

- with geometrical imperfections corresponded to the global buckling mode;

The global imperfections were considered with a sinusoidal shape given by the expression (7). An initial rotation with maximum of L/1000 rad was also used, proximally following the LTB mode shape.

$$y = \frac{L}{1000} \sin\left(\frac{\pi x}{L}\right) \qquad (7)$$

The maximum value for the local imperfections was of b/200 (being b the width of the plate) [16].

Figure 9 shows a 460x150x3x4 beam with length of 3400 mm with the different introduced geometrical imperfections.

In each set of results, obtained for 500 ºC (see Figure 10), a small plateau on the numerical results was obtained. It is observed that the beams without any imperfections reach higher resistances compared to the ones which take into account initial imperfections. In this graph it can be noticed the influence of the introduction of different geometrical imperfections. For large slenderness the analysis with global imperfections give lower ultimate loads when compared with the other simulations, while for low slenderness the local imperfections are the ones that reduce the obtained ultimate loads.

According to Part 1-5 of EC3 [16], a combination of the previous enunciated geometrical imperfections should be introduced on the numerical model. According to this norm, in combining imperfections a leading imperfection should be chosen and the accompanying imperfections may have their values reduced to 70%. A first test was made with simply adding both imperfections without applying the above mentioned proposed combination. The local and the

global imperfections gave the lowest bending resistance moments, for small and large beams lengths respectively, always very close to the simply adding combination, meaning that there is no significant influence from considering the combination with the reduction of 70% of the accompanying imperfections. Therefore the possibility of simply adding the imperfections (see Figure 11) was chosen to be applied on the parametric study.

5.2   Influence of the residual stresses

In this section it is presented an evaluation of the influence of residual stresses in stainless steel beams at high temperatures.

The adopted residual stresses follow, the typical patterns for carbon steel welded sections [17, 18], considered constant across the thickness of the web and flanges. The distribution is shown in Figure 12, and has the maximum value of fy (yield strength). On the figure it is possible to recognize the principal stresses corresponded to the introduced residual stresses of tension and compression.

Figure 13 shows the numerical results obtained for the beams 460x250x3x4 at 500 ºC with and without residual stresses. It can be concluded that there are differences suggesting that the residual stresses should be considered, as it was made on the parametric study.

6   Parametric study

In this parametric study, Figure 14 to Figure 22 compare the curves obtained using Part 1.2 of EC3, French national annex of this EN 1993-1-2 and the EuroInox manual proposal, described in section 3 of this paper. To highlight the accuracy and safety of the proposals Figure 16, Figure 19 and Figure 22 are presented indicating the level of approximation to the numerical results of each proposal.

From the graphs it can be concluded that EN 1993-1-2 approach is always on the safe side being most of the time too conservative. The French national annex gives results that are unsafe for the grades 1.4301 and 1.4003, for the grade 1.4462 the results are in good agreement with the numerical simulations. The EuroInox manual is the one that approximates better the numerical results from SAFIR, having however some unsafe results for the grade 1.4301 and some too conservative results for the grade 1.4462.

The plateau on the buckling curves at slenderness of 0.2, as proposed on the French national annex, seems to fit better with the grades 1.4301 and 1.4003, while for the grade 1.4462 the plateau of 0.4 is more appropriate.

These results lead to believe that some small adjustment on the presented proposals could be made improving the accuracy and safety of these design procedures.

7   Conclusions

In this work it was presented a numerical study on the behaviour of thin walled stainless steel I beams with Class 4 cross sections in case of fire.

The elastic critical loads and buckling modes of those profiles were determined through CUFSM program. From this analysis it was possible to better predict the behaviour of the analysed beams and to apply those buckling modes shapes on the initial geometric imperfections considered on the numerical parametric analysis.

The influence of initial geometrical imperfections (local, global, and both added) on the determination of the ultimate loads of these elements at high temperatures was analysed. It was concluded that these imperfections are relevant to the determination of those ultimate loads, and that they should be considered the expected collapse mode.

Finally, it was also performed a comparison between the obtained finite element results from the program SAFIR for the determination of ultimate loads and different design approaches as the requirements prescribed in Part 1-2 of Eurocode 3, the French national annex of the same Part of the EC3 and the EuroInox manual proposal, concluding that some small adjustments could be made to each of the proposals to improve the approximations to the numerical results.

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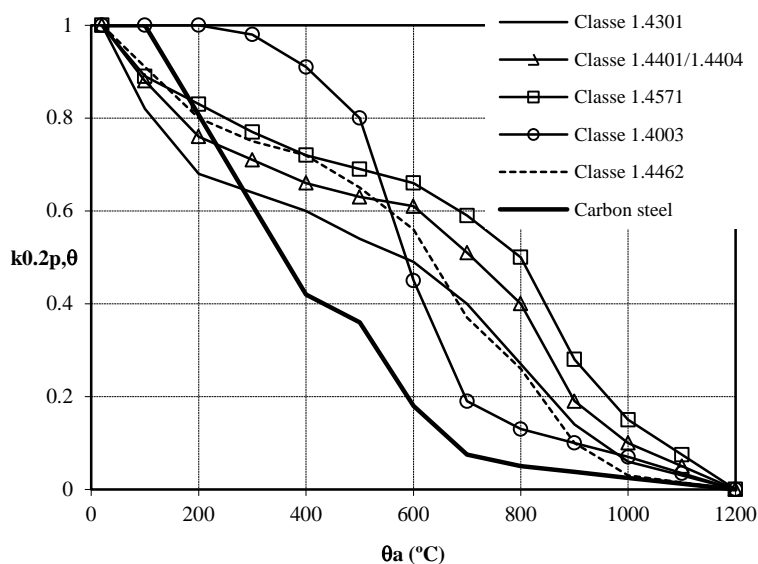
Figure 1   **Comparison between the strength reduction factor at high temperatures of the several stainless steel grades and carbon steel.**
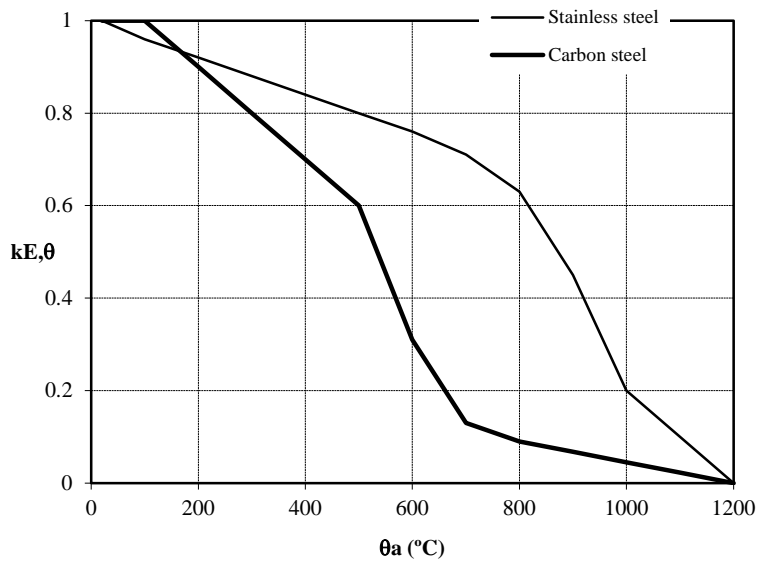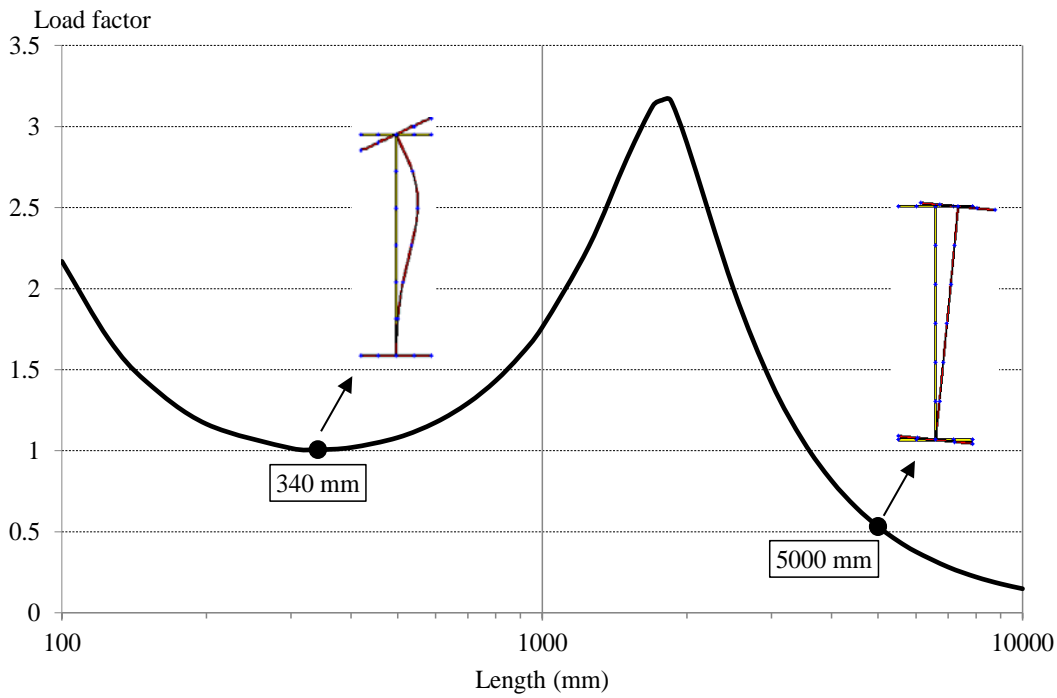
Figure 2 Comparison between the elasticity modulus reduction factor at high temperatures of the stainless steel and carbon steel.
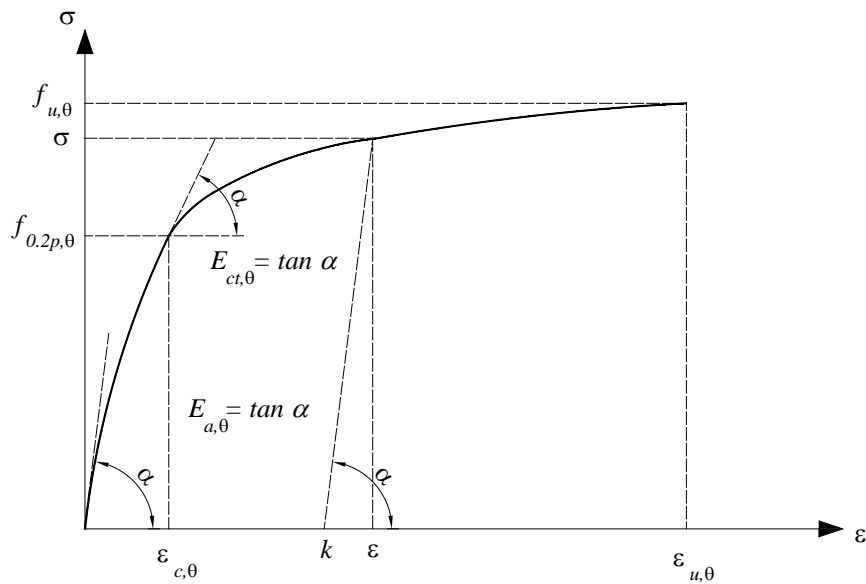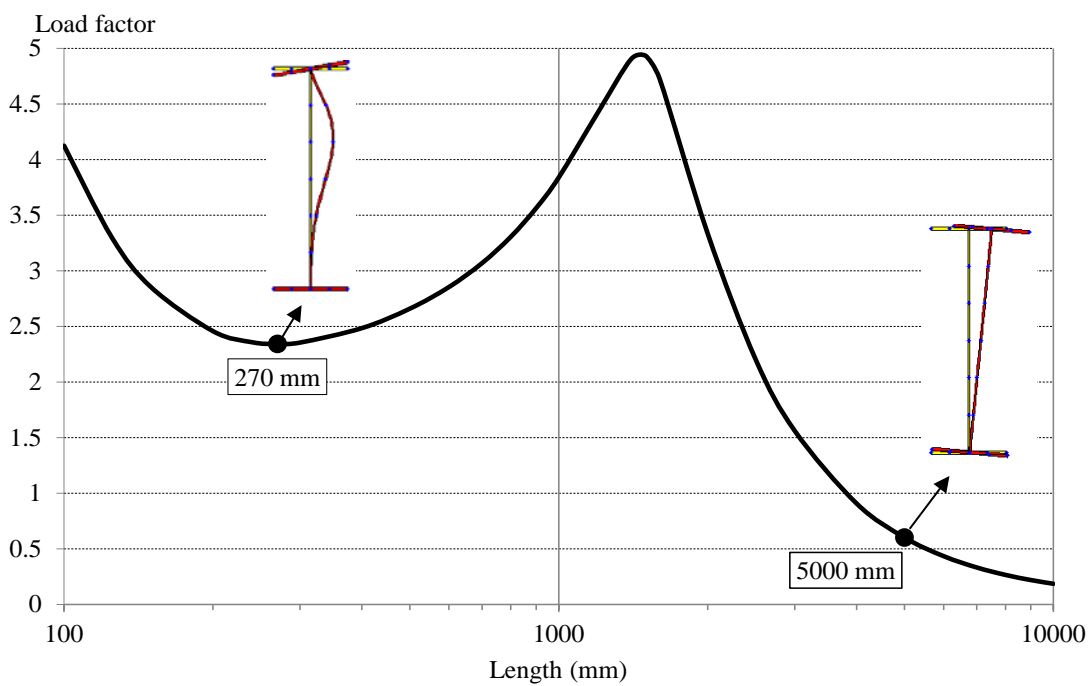

Figure 3 Stress-strain relationship of the stainless steel at elevated temperatures
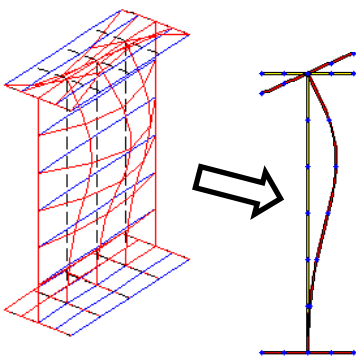
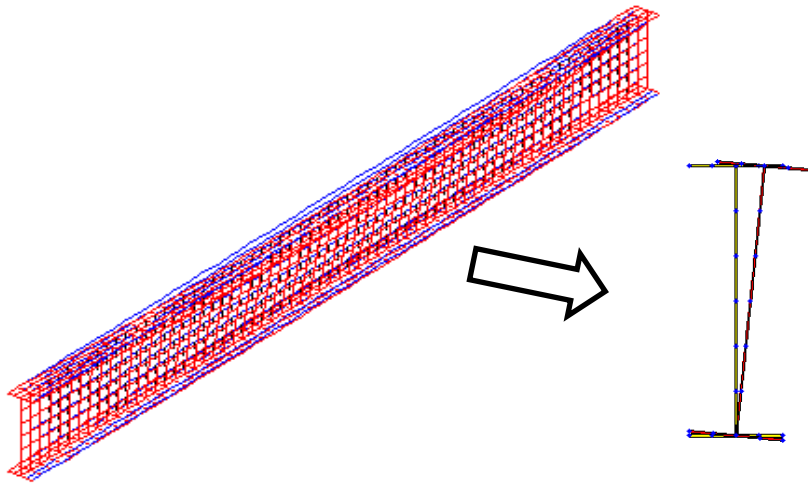**Figure 4**　　Buckling mode instability curve of a beam with the cross-section 460x150x3x4.

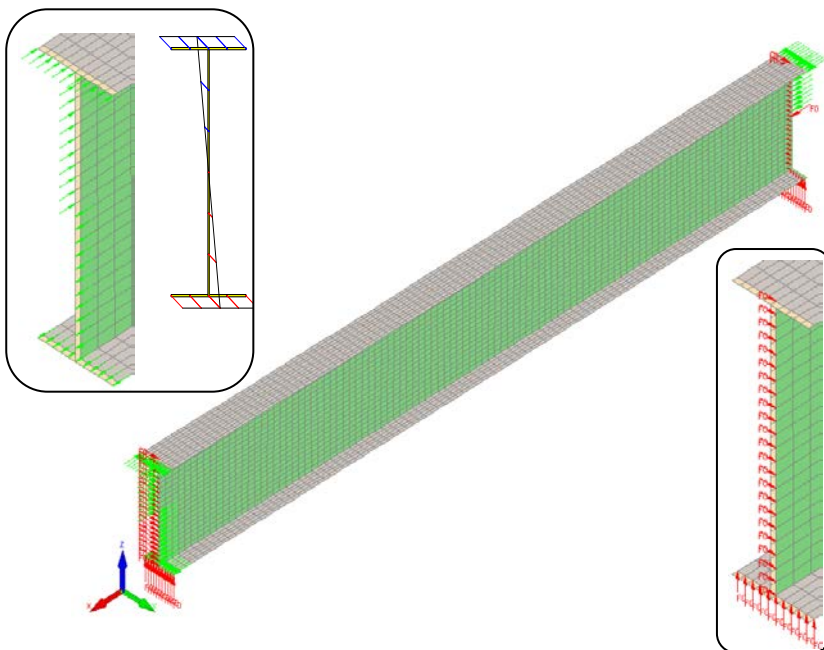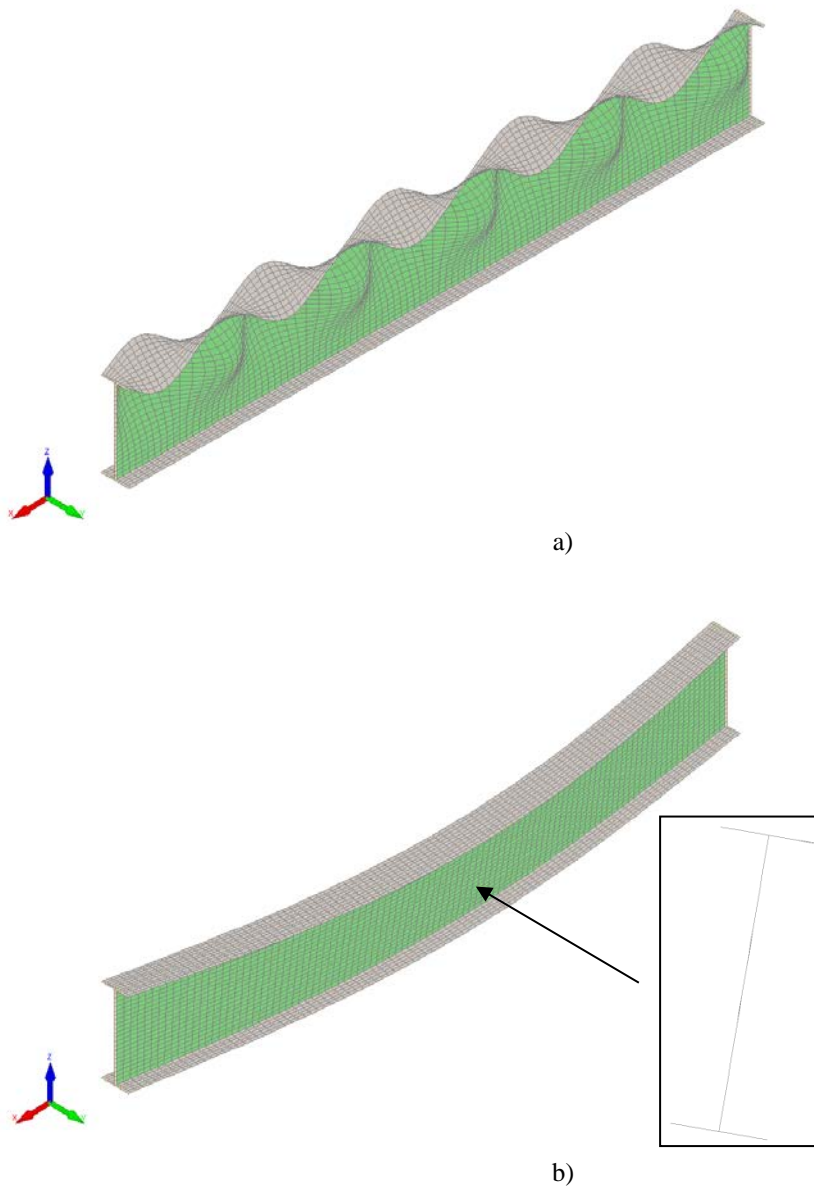**Figure 5**　　Buckling mode instability curve of a beam with the cross-section 460x150x4x7.

Figure 6　　Buckling mode shape of a beam with the cross-section 460x150x3x4 and 340 mm.

Figure 7   **Buckling mode shape of a beam with the cross-section 460x150x3x4 and 5000 mm.**

Figure 8   **Numerical model, restrictions, and applied loads and mesh size.**

a)

b)

Figure 9   **Considered geometric imperfections (amplified 50 times): a) local; b) global.**
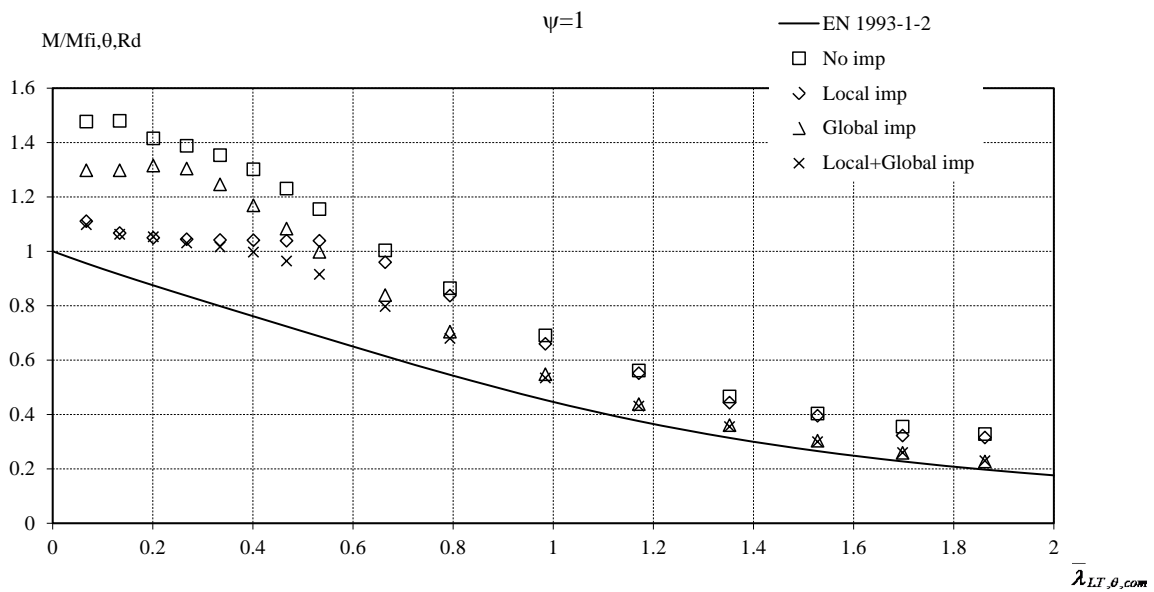
Figure 10   **Influence of the geometric imperfections on the numerical results.**
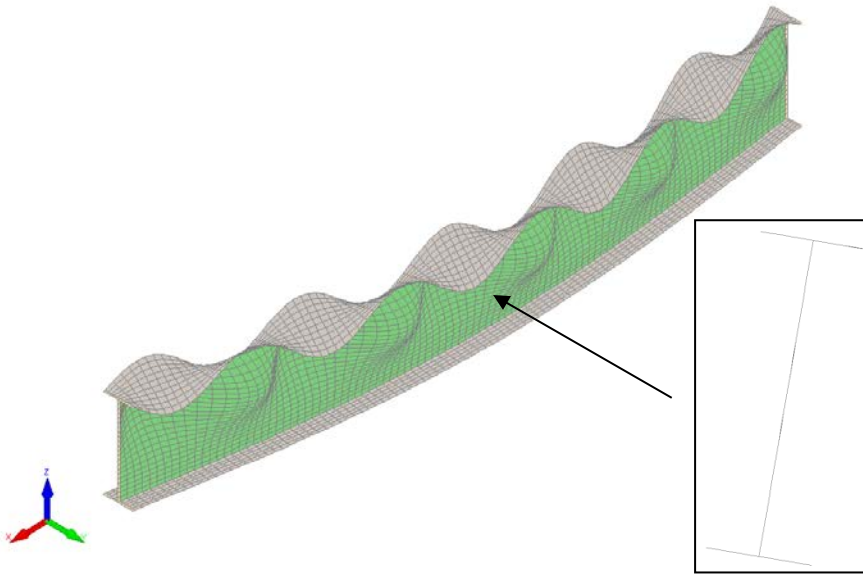
Figure 11   Beam with local plus global imperfections.
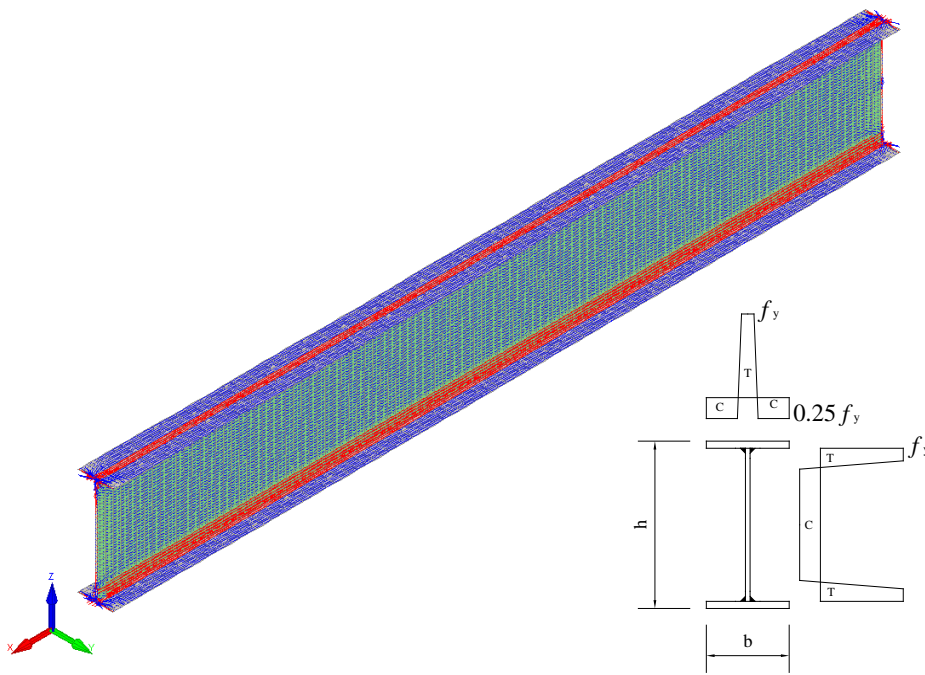
Figure 12   Residual stresses patterns on welded profiles.
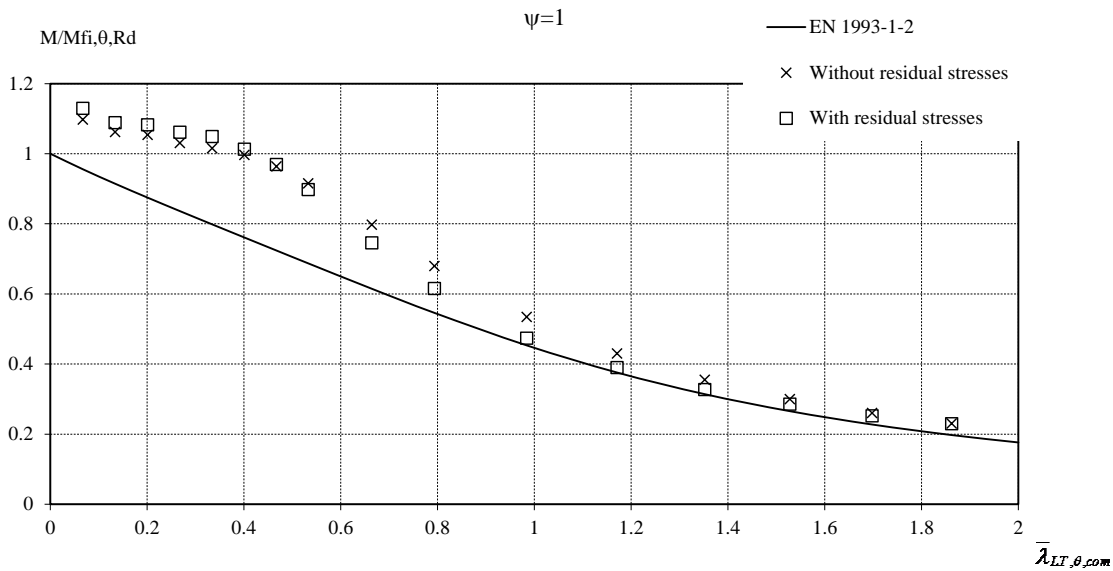
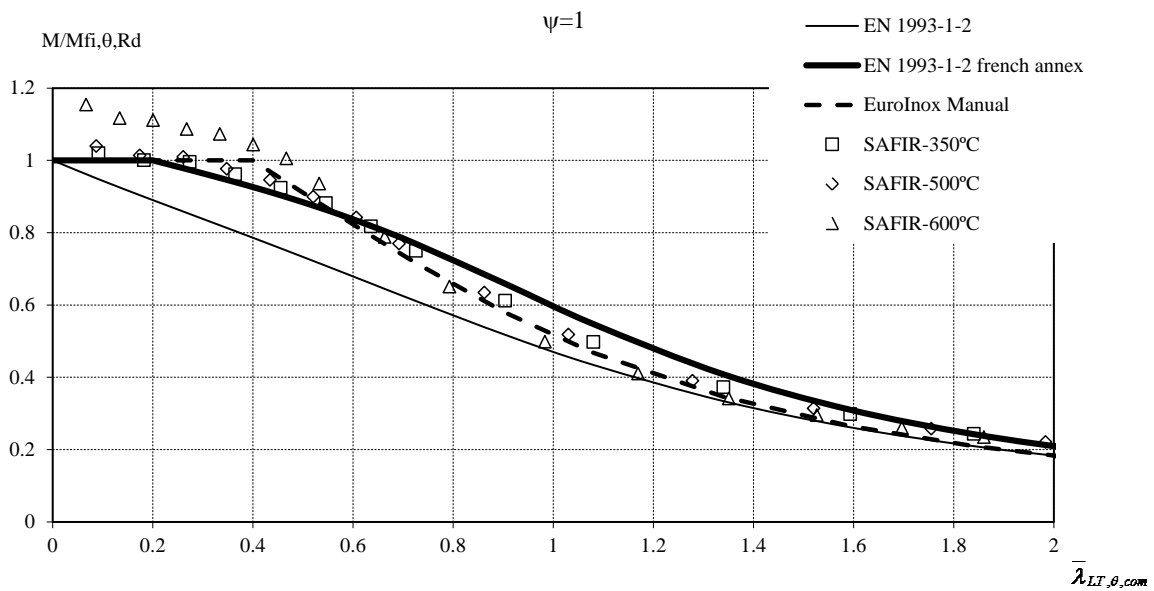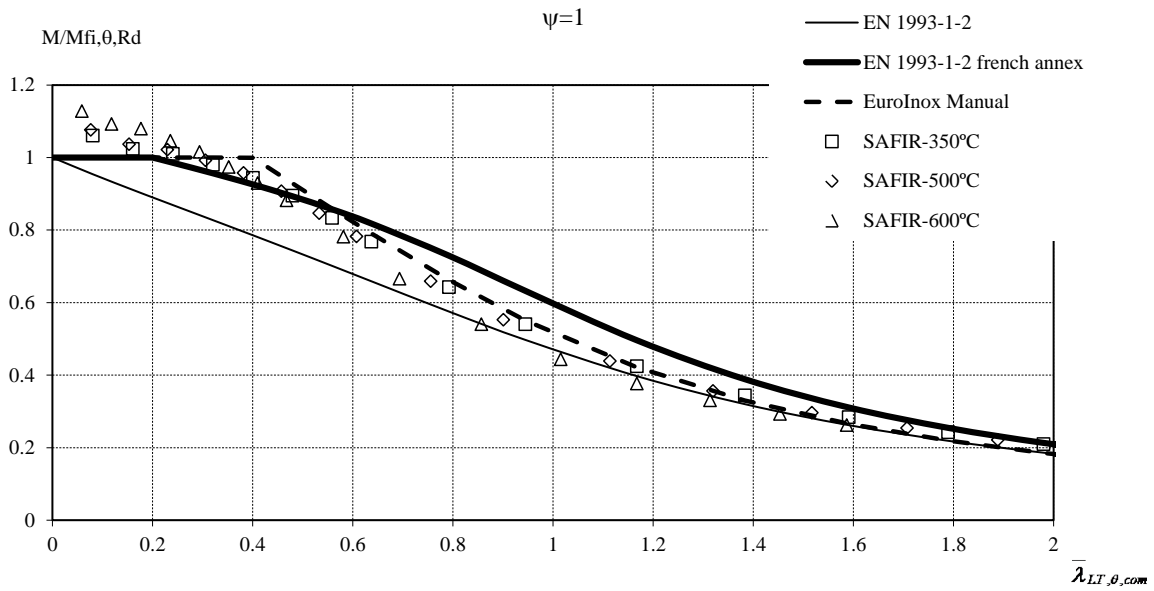**Figure 13**   **Influence of the residual stresses on the numerical results.**
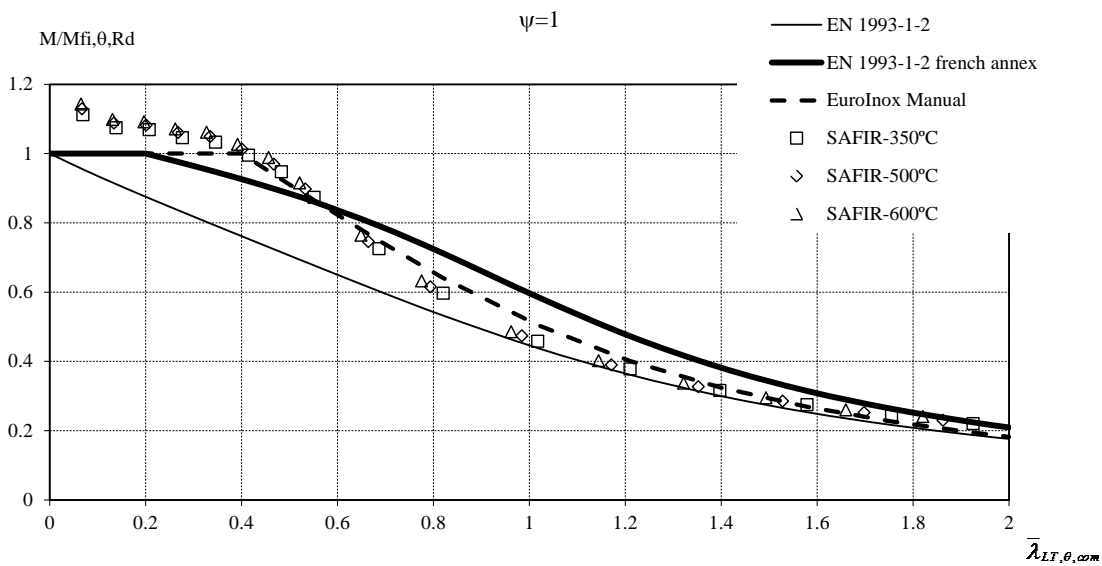
Figure 14   **Stainless steel beams 460x150x3x4 of the grade 1.4301.**
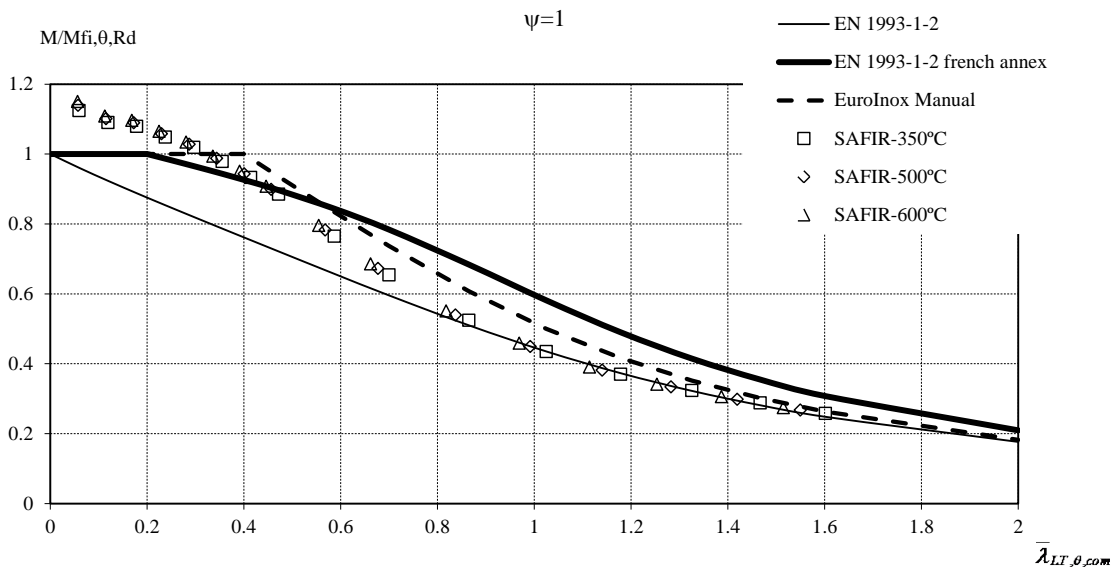
Figure 15   **Stainless steel beams 460x150x4x7 of the grade 1.4301.**
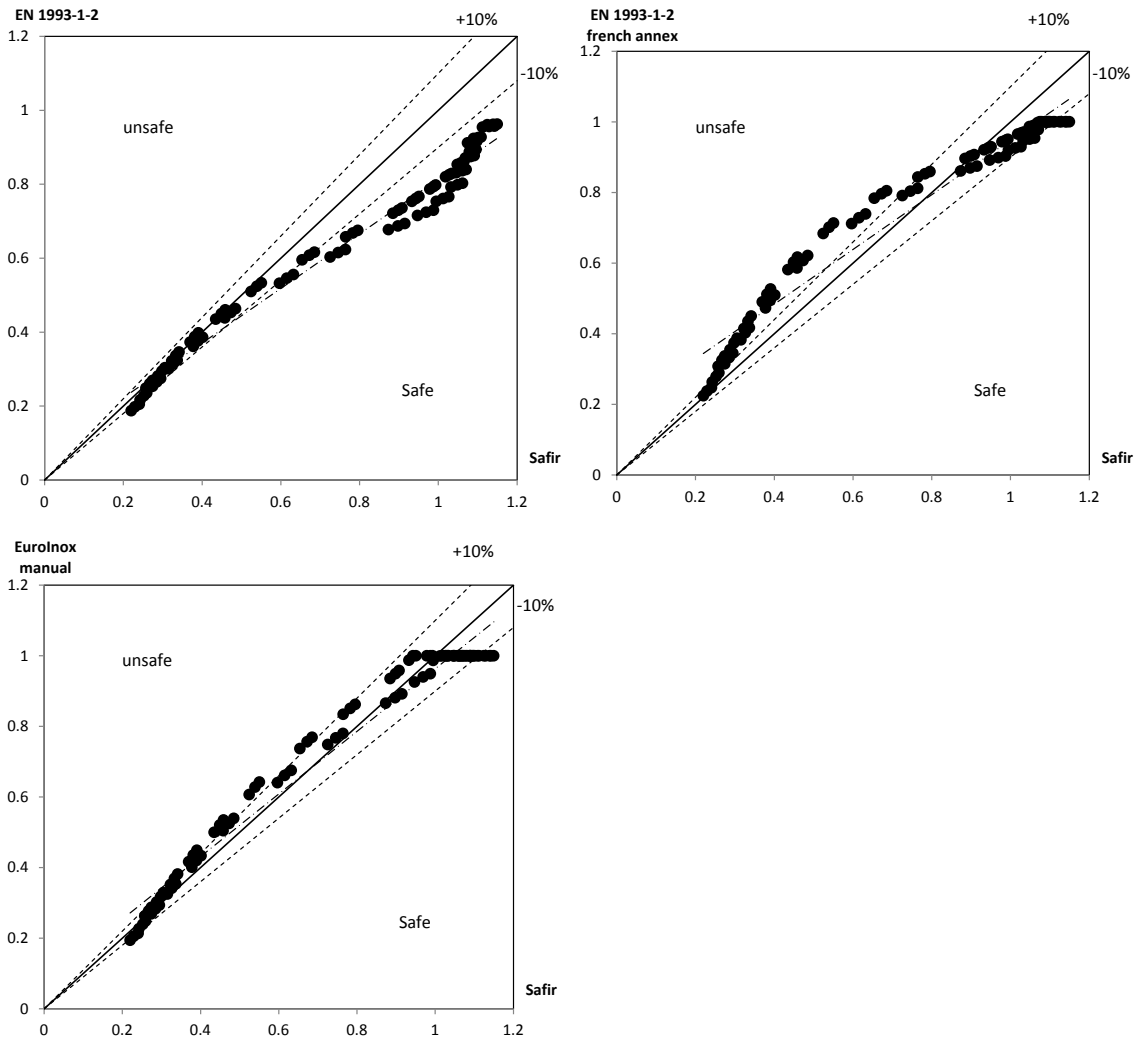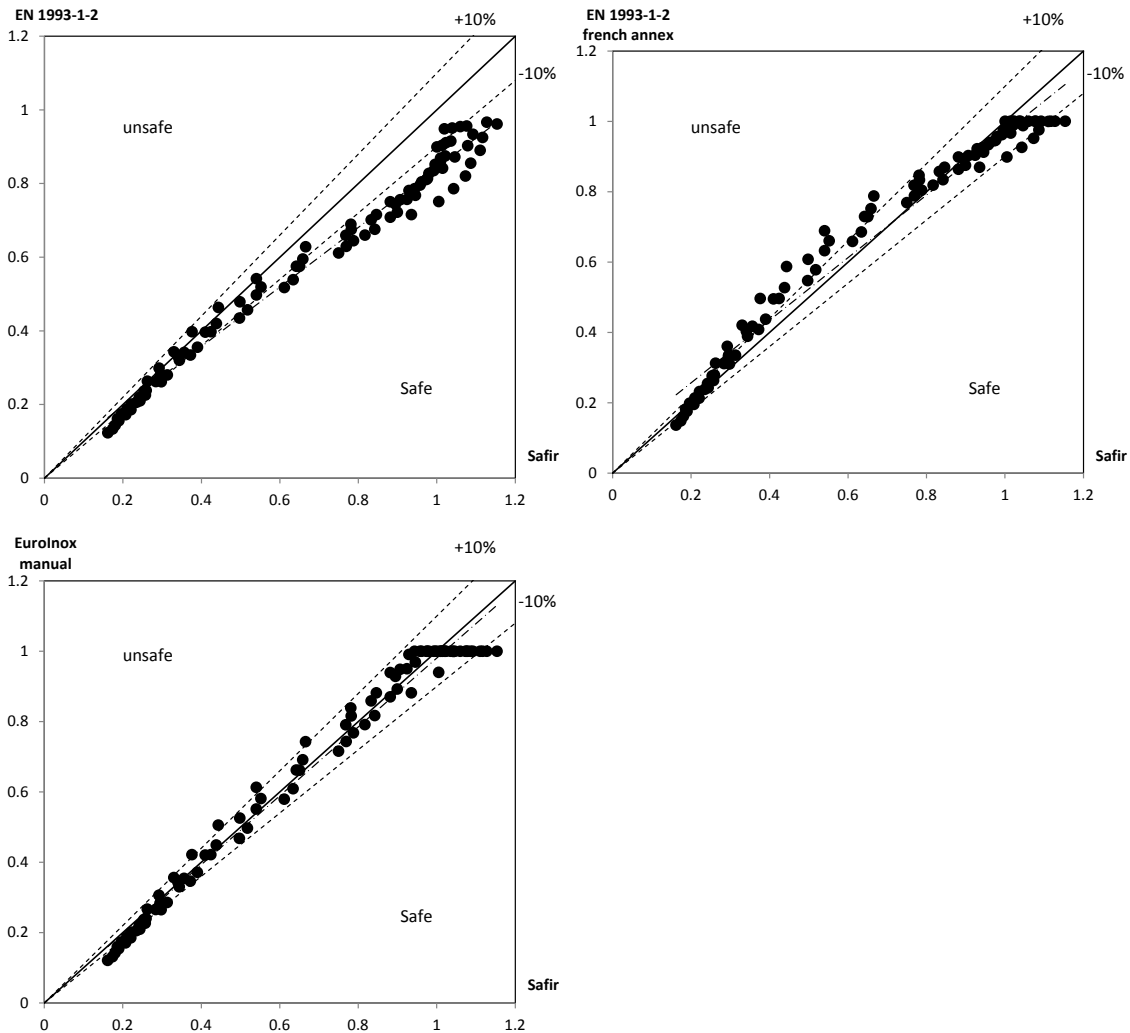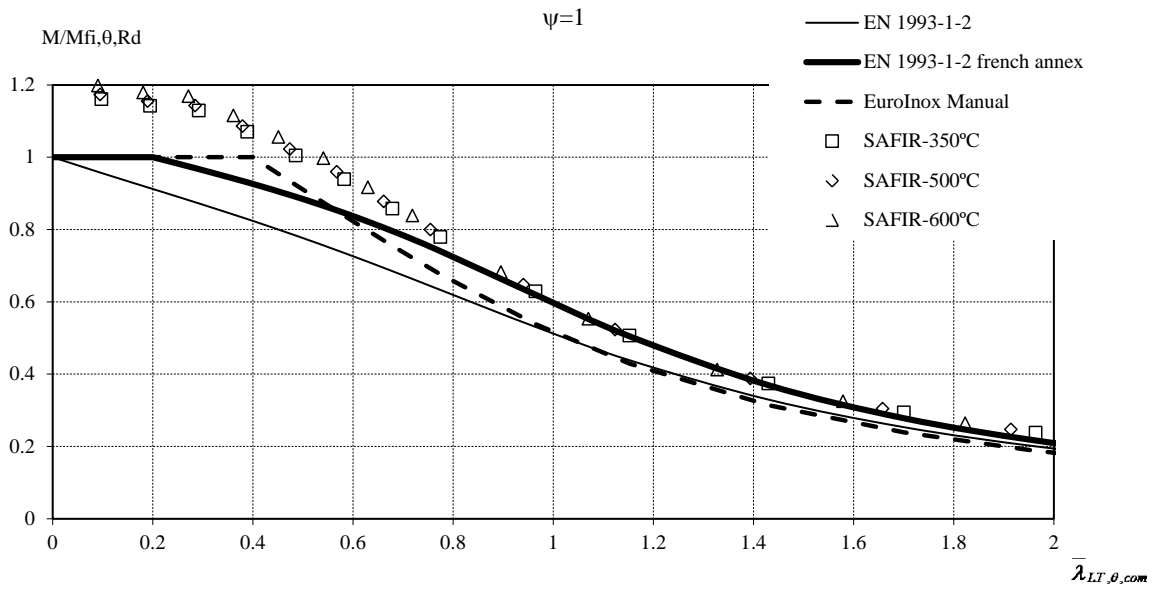

Figure 16   **Both stainless steel beams of the grade 1.4301.**

Figure 17   **Stainless steel beams 460x150x3x4 of the grade 1.4003.**

$\psi=1$

M/Mfi,θ,Rd

$\overline{\lambda}_{LT,\theta,com}$

- EN 1993-1-2
- EN 1993-1-2 french annex
- EuroInox Manual
- □ SAFIR-350ºC
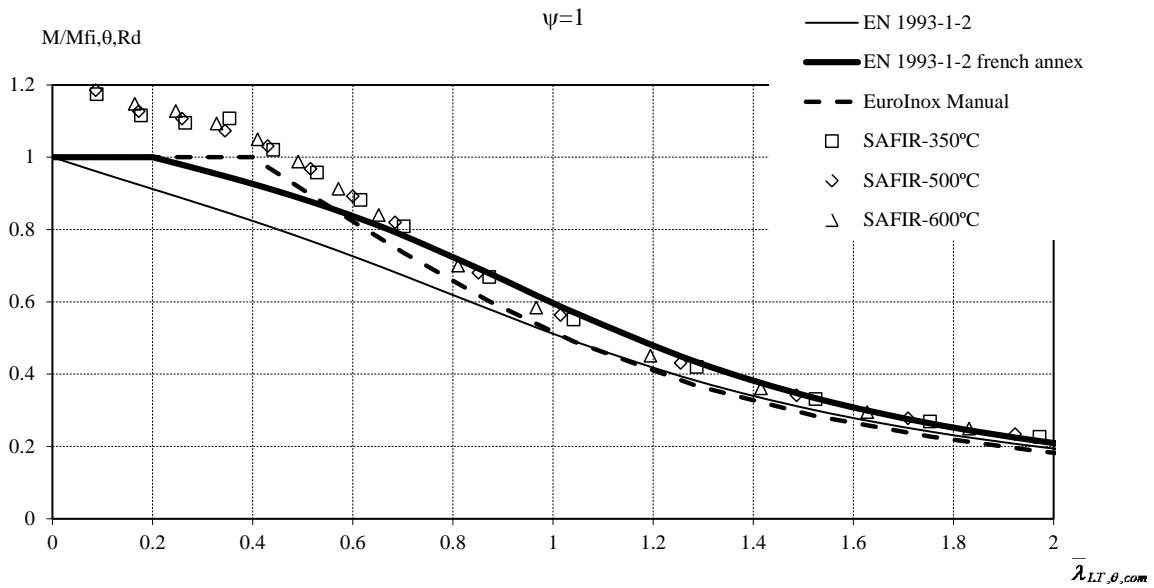- ◇ SAFIR-500ºC
- △ SAFIR-600ºC

Figure 18 **Stainless steel beams 460x150x4x7 of the grade 1.4003.**

Figure 19 **Both stainless steel beams of the grade 1.4003.**

Figure 20   Stainless steel beams 460x150x3x4 of the grade 1.4462.

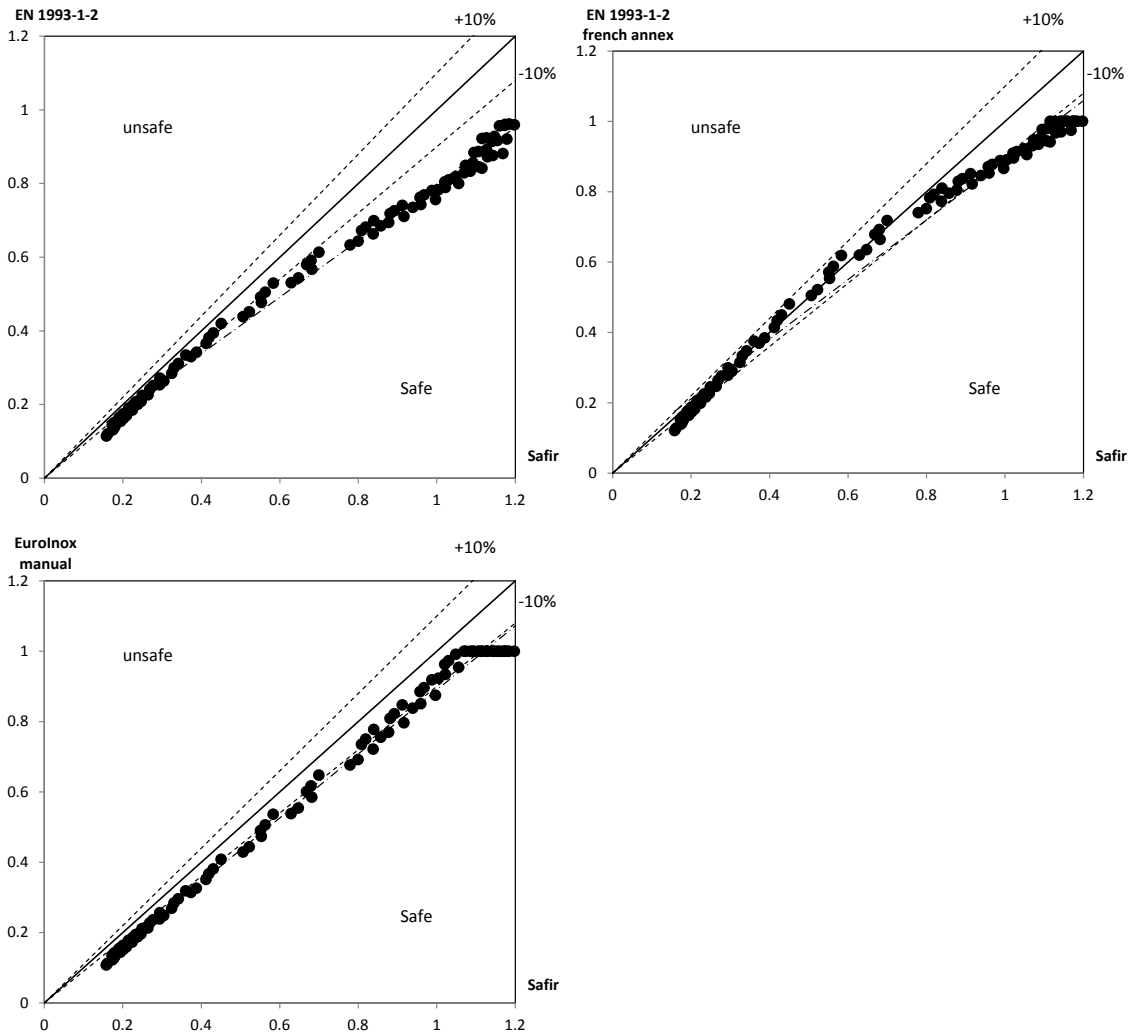**Figure 21   Stainless steel beams 460x150x4x7 of the grade 1.4462.**

Figure 22   **Both stainless steel beams of the grade 1.4003.**

Table 1   **Expressions of the constitutive law of the stainless steel at high temperatures [5]**

Strain range	Stress σ	Tangent modulus
$\varepsilon \le \varepsilon_{c,\theta}$	$\dfrac{E \cdot \varepsilon}{1 + a \cdot \varepsilon^b}$	$\dfrac{E\left(1 + a \cdot \varepsilon^b - a \cdot b \cdot \varepsilon^b\right)}{\left(1 + a \cdot \varepsilon^b\right)^2}$
$\varepsilon_{c,\theta} < \varepsilon < \varepsilon_{u,\theta}$	$f_{0.2p,\theta} - e + (d/c)\sqrt{c^2 - \left(\varepsilon_{u,\theta} - \varepsilon\right)^2}$	$\dfrac{d\left(\varepsilon_{u,\theta} - \varepsilon\right)}{c\sqrt{c^2 - \left(\varepsilon_{u,\theta} - \varepsilon\right)^2}}$
Additional parameters and functions are given in Annex C of EN 1993-1-2		