SDSS'Rio 2010 STABILITY AND DUCTILITY OF STEEL STRUCTURES
E. Batista, P. Vellasco, L. de Lima (Eds.)
Rio de Janeiro, Brazil, September 8 - 10, 2010

NUMERICAL STUDY ON STAINLESS STEEL BEAM-COLUMNS WITH TRANSVERSE LOADING

N. Lopes*, P. Vila Real* and L. Simões da Silva**

* LABEST – Department of Civil Engineering, University of Aveiro, Portugal
e-mails: nuno.lopes@ua.pt, pvreal@ua.pt

** ISISE – Department Civil Engineering, University of Coimbra, Portugal
e-mail: luisss@dec.uc.pt

Keywords: Stainless Steel, Beam-columns, Eurocode 3, Structures, Numerical.

Abstract. *In this work the accuracy and safety of the currently prescribed design rules in Eurocode 3 (EC3), for the resistance of stainless steel uniform beam-columns with transverse loading, are evaluated. This study was performed through numerical tests obtained using the methodology usually named by GMNIA (geometrically and materially non-linear imperfect analysis). This evaluation is carried out on members with Class 1 and Class 2 cross-sections subjected to axial compression and bending produced by in-plane transverse loading (concentrated and distributed loads perpendicular to the member axis on both the strong or weak axis). In these studies, the possibilities of occurring or not lateral-torsional buckling (LTB) are both analysed. The obtained numerical results are compared with the interaction curves obtained from Part 1-4 of EC3 and from other proposed design approaches for beam-columns.*

1 INTRODUCTION

Recent research studies [1, 2, 3] have been acknowledging the stainless steel important mechanical properties, contributing at the same time for the widespread and increase of structural applications with this type of steel. Nevertheless it is still recognized the necessity of a better characterization of its structural behaviour.

For stainless steel structural elements design subject to combined bending and axial compression, Part 1-4 of EC3 [4] has two notes saying that the national annexes may give other interaction formulae and others interaction factors, which suggests that the beam-columns formulae and the interaction factors were not well established for stainless steel members, at the time of the conversion from ENV to EN.

Regarding conventional carbon steel beam-column design, Part 1-1 of EC3, EN 1993-1-1 [5], introduced several changes in the design formulae for carbon steel beam-columns, when compared to the ENV version of EC3. Two new methods for the design of carbon steel beam-columns at room temperature are proposed in Part 1-1 of EC3 [5], which are the result of the work carried out by two working groups that followed different approaches [7], a French-Belgian team and an Austrian-German one. In this paper it will be checked if these two procedures can also be used with stainless steel members.

In recent research work [8], parametric studies of the behaviour of beam-columns in several stainless steel grades (austenitics, austenitic-ferritic and ferritics grades), with only end moments (uniform, triangular and bi-triangular bending moment diagrams) and an axial compression, were performed, resulting in the presentation of new proposals for the design of these structural elements. In these studies [8] it was evaluated the possibility of having, in parts 1-1 and 1-4 of the EN versions of EC3, the same approach for beam-columns design. This consideration of the two methods in Part 1-1 for stainless steel structural elements design is also studied in this paper.

In addition, previous papers from the authors [9, 10], have presented new proposals for the flexural buckling of stainless steel columns and LTB of stainless steel beams, which are more accurate than the prescriptions in EC3 and safe when compared with the numerical results. These new proposals necessarily affect the behaviour of the interaction formulae for beam-columns and their influence were taken into account in the present paper.

In this work it is evaluated the accuracy and safety, on the resistance calculation of stainless steel beam-columns with and without LTB, subjected to transverse loads, of the: currently prescribed design rules in Part 1-4 of EC3; new carbon steel beam-column formulae at room temperature [4, 7], after being adapted to deal with stainless steel; and the recently proposed design expressions for stainless steel beam-columns with only end moments [8].

This evaluation is carried out by performing numerical simulations on Class1 and Class 2 stainless steel H-columns subjected to compression plus uniaxial bending. It is considered buckling in the two main cross-section axes and different bending moment diagrams (point load applied at the mid span and distributed load along the member length). HEA200 welded cross-sections were used of stainless steel grade 1.4301. The stainless steel mechanical properties used in this work can be found in Part 1-4 of EC3 [4]. An average of 5 beam lengths and 8 bending moment / axial load ratios were analyzed for each case.

In the numerical simulations, a sinusoidal lateral geometric imperfection was considered [11]. The adopted residual stresses follow the typical pattern for carbon steel welded sections [11, 12, 13], considered constant across the thickness of the web and flanges.

The numerical results were obtained with the finite element program SAFIR [14], a geometrical and material non linear finite element code, which has been adapted to be able to model stainless steel structures according the material properties defined in EN 1993-1-4.

The safety factor γ_{M1} in stainless steel structures at room temperature takes the value of 1.1 [4]. However in order to compare the actual interaction curves, in the studies presented in this paper, this factor was considered equal to 1.0, as already adopted in carbon steel design [5].

2 BEAM-COLUMN WITHOUT LTB

In this work, the possibility of the beam-columns not having LTB was first studied. It was only considered axial compression: with bending in the strong axis, assuming that the element is restrained about the z axis; and with bending in the weak axis, assuming that the element is restrained about the y axis.

2.1 EC3 proposal

Part 1-4 of EC3, gives the following expressions for the design of Class 1 and 2 stainless steel beam-columns subjected to axial compression and bending without LTB.

$$\frac{N_{Ed}}{N_{b,Rd,\min}}+k_y\frac{M_{y,Ed}}{W_{pl,y}\dfrac{f_y}{\gamma_{M1}}}+k_z\frac{M_{z,Ed}}{W_{pl,z}\dfrac{f_y}{\gamma_{M1}}}\leq 1 \tag{1}$$

where

$$k_i=1.0+2\left(\overline{\lambda}_i-0.5\right)\frac{N_{Ed}}{N_{b,Rd,i}}\quad \text{but}\quad 1.2\leq k_i\leq 1.2+2\frac{N_{Ed}}{N_{b,Rd,i}} \tag{2}$$

being *i* *y* or *z* axis

As it can be observed these formulae do not consider the influence of the shape of the bending moment diagram.

2.2 Adaptation of the carbon steel interaction curves

As mentioned before, the project team involved in the conversion from ENV to EN of Part 1-1 of EC3 has revised the interaction formulae for carbon steel beam-columns safety check [5]. Two new methods were incorporated: “Method 1” and “Method 2” reported in Annex A and Annex B of Part 1-1 of EC3, respectively. These two methods were adapted to take into account the reduction factor for flexural buckling of stainless steel columns [9].

According to EN 1993-1-1, the stability of beam-columns (of the classes 1 and 2), in the case of bending around the strong and weak axis, is checked in accordance with the following interaction formulae:

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\frac{M_{y,Rk}}{\gamma_{M1}}} \leq 1 \text{ and } \frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \tag{3}$$

where k_{yy} and k_{zz} are the interaction factors.

2.2.1 Method 1

Without attempting to explain the background [7] of this proposal, the interaction factors are expressed through the relations, which use the equivalent moment factors:

$$c_{mi} = c_{mi,0} \tag{4}$$

$c_{mi,0}$ is respectively, for a mid span point load and for a uniformly distributed load:

$$c_{mi,0} = 1 - 0.18 \frac{N_{Ed}}{N_{cr,i}} \text{ and } c_{mi,0} = 1 + 0.03 \frac{N_{Ed}}{N_{cr,i}} \tag{5}$$

The other coefficients not described here can be obtained in Annex A of Part 1-1 of EC3

2.2.2 Method 2

“Method 2” is described in Annex B of Part 1-1 of EC3. This approach proposes different interaction factors from “Method 1”, using, for a mid span point load and for uniformly distributed load respectively

$$c_{mi} = 0.90 \text{ and } c_{mi} = 0.95 \tag{6}$$

2.3 Formulation of a new proposal

Part 1-4 of EC3 [4] states that the safety evaluation of elements subjected to bending and axial compression should satisfy expression (1).

Based in the procedure in [15] for the determination of the carbon steel interaction curves at high temperatures, new formulae, for the stainless steel beam-columns safety evaluation, were developed [8], following the approaches included in the ENV version of Part 1-1 of the same EC3 [6] and adopted in Part 1-2 of EC3 [16]. In comparison with EC3 [4] the main changes appear in the determination of the interactions factors k_y and k_z.

$$k_i = 1 - \frac{\mu_i N_{Ed}}{\chi_i A \frac{f_y}{\gamma_{M1}}} \quad \text{with} \quad k_i \leq 1.5 \text{ and } k_i \geq \mu_y - 0.7 \tag{7}$$

To determine the values of μ_y and μ_z the following equations should be used

$$\mu_y = \left(0.97\beta_{M,y} - 2.11\right)\overline{\lambda}_y + 0.44\beta_{M,y} + 0.09 \qquad \text{if } \overline{\lambda}_y \leq 0.3 \text{ then } \mu_y \leq 1.0 \text{ else } \mu_y \leq 0.9 \tag{8}$$

$$\mu_z = \left(1.09\beta_{M,z} - 2.32\right)\overline{\lambda}_z + 0.29\beta_{M,z} + 0.48 \qquad \text{if } \overline{\lambda}_z \leq 0.3 \text{ then } \mu_z \leq 1.0 \text{ else } \mu_z \leq 0.9 \tag{9}$$

Finally the equivalent uniform moment factors $\beta_{M,y}$ and $\beta_{M,z}$ can be determined in function of the bending diagram shape as in [6] and [16], according to

$$\beta_M = \beta_{M,\psi} + \frac{M_Q}{\Delta M}\left(\beta_{M,Q} - \beta_{M,\psi}\right) \tag{10}$$

where $M_Q, \Delta M, \beta_{M,Q}$ and $\beta_{M,\psi}$ can be found in part 1-2 of EC3 [16].

For a mid span point load and for uniformly distributed load its value is respectively

$$\beta_{M,i} = \quad 4 \text{ and } \beta_{M,i} = 1.3 \tag{11}$$

2.4 Accuracy of the proposals

In this section the new proposals are evaluated by means of a direct comparison with numerical results. Due to space limitations only few results of the parametric study are shown.

The graphics from Figure 1 and Figure 2 were obtained for beam-columns with buckling about y and z axis, with uniaxial bending in the strong and weak axis respectively. Here, the length of 3 m corresponds to a non-dimensional slenderness values of $\bar{\lambda}_y = 0.37$ and $\bar{\lambda}_z = 0.62$, while the length of 7 m corresponds to $\bar{\lambda}_y = 0.87$ and $\bar{\lambda}_z = 1.45$.

The interaction curves in the graphics are obtained from: Part 1-4 of EC3 “EN 1993-1-4”; Part 1-4 of EC3 with a new proposal for columns [9] “EN 1993-1-4 mod”; Part 1-1 of EC3 for carbon steel beam-columns with a new proposal for columns [9] “Method 1”and “Method 2” and the interaction curves presented in the previous section [8] “New proposal”.

The method which better approximates the numerical results from SAFIR is the “New proposal”. “Method 1” and “Method 2” adapted from the formulae from Part 1-1 of EC3 for carbon steel and the new proposal for stainless steel columns also present good approximations. From these two methods, the one that has a better behaviour is “Method 1”, but still not as good as the “New proposal”. It can also be observed that the new proposal for columns [9] introduces a significant improvement in the interaction curves approximations to the numerical results.

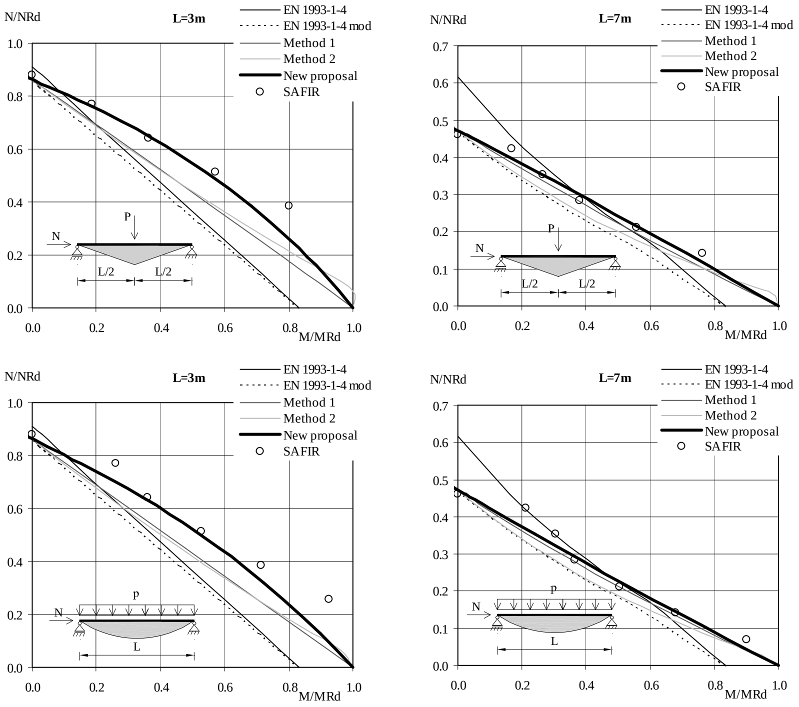

Figure 1: Different interaction curves, regarding compression and uniaxial bending about the strong axis.

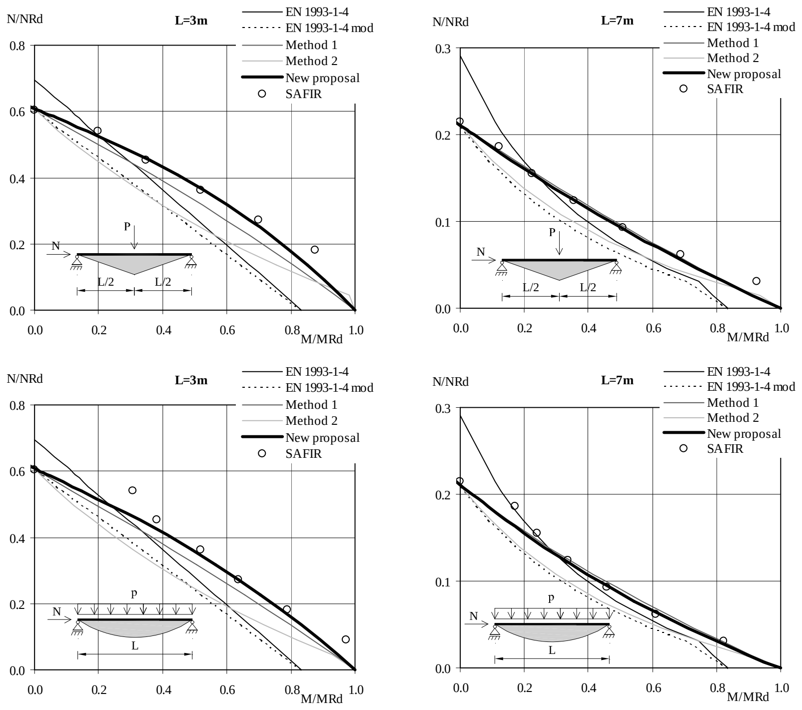

Figure 2: Different interaction curves, regarding compression and uniaxial bending about the weak axis.

3 BEAM-COLUMN WITH LTB

In this section it is presented the study made on stainless steel beam-columns with LTB. It was only considered axial compression with bending in the strong axis, assuming that the element is only restrained in the extremities by fork supports.

3.1 EC3 proposal

Part 1-4 of EC3, gives the following expressions for the design of Class 1 and 2 stainless steel beam-columns subjected to axial compression and bending, having the possibility of occurring LTB.

$$\frac{N_{Ed}}{\left(N_{b,Rd}\right)_{\min}} + k_{LT}\frac{M_{y,Ed}}{M_{b,Rd}} + k_z\frac{M_{z,Ed}}{W_{pl,z}\dfrac{f_y}{\gamma_{M1}}} \leq 1 \tag{12}$$

where

$$k_{LT} = \ 0 \ . \ 1 \tag{13}$$

and k_z is given by expression (2).

3.2 Adaptation of the carbon steel interaction curves

As made before, it was studied the possibility of using the interaction curves recommended in Part 1-1 of EC3 [5], adapted to the stainless steel material properties. The two already described methods were also changed in order to account for the reduction factor for flexural buckling of stainless steel columns and the reduction factor for LTB of stainless beams.

According to EN 1993-1-1, the stability of beam-columns with LTB (of the class 1 and 2), in the case of bending around the strong axis, is checked in accordance with the following interaction formulae:

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} \le 1 \text{ and } \frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} \le 1 \tag{14}$$

k_{yy} and k_{zy} are interaction factors different from the one prescribed for beam-columns without LTB.

3.2.1 Method 1

The procedure for the determination of the interaction factors for the "Method 1" is reported in Annex A of Part 1-1 of EC3, where:

$$c_{my} = c_{my,0} + \left(1 - c_{my,0}\right) \frac{\sqrt{\varepsilon_y}\, a_{LT}}{1 + \sqrt{\varepsilon_y}\, a_{LT}} \text{ and } c_{mLT} = c_{my}^2 \frac{a_{LT}}{\sqrt{\left(1 - \frac{N_{Ed}}{N_{cr,z}}\right)\left(1 - \frac{N_{Ed}}{N_{cr,T}}\right)}} \ge 1 \tag{15}$$

3.2.2 Method 2

"Method 2" is described in Annex B of Part 1-1 of EC3. According to this method, for a mid span point load and for uniformly distributed load respectively

$$c_{mLT} = c_{my} = 0.90 \text{ and } c_{mLT} = c_{my} = 0.95 \tag{16}$$

3.3 Formulation of a new proposal

In the reference [8] it is proposed that the safety evaluation of elements subjected to bending and axial compression with LTB should satisfy:

$$\frac{N_{Ed}}{\chi_z A \frac{f_y}{\gamma_{M1}}} + k_{LT} \frac{M_{y,Ed}}{\chi_{LT} W_{pl,y} \frac{f_y}{\gamma_{M1}}} + k_z \frac{M_{z,Ed}}{W_{pl,z} \frac{f_y}{\gamma_{M1}}} \le 1 \tag{17}$$

In these proposed interaction curves the interactions factors k_{LT} should be determined by

$$k_{LT} = 1 - \frac{\mu_{LT} N_{Ed}}{\chi_z A \frac{f_y}{\gamma_{M1}}} \quad \text{with} \quad k_{LT} \le 1 \text{ and } k_{LT} \ge \mu_{LT} - 0.7 \tag{18}$$

where

$$\begin{gathered} \mu_{LT} = \left(-0.07\beta_{M,LT} - 0.07\right)\bar{\lambda}_z + 0.60\beta_{M,LT} - 0.10 \\ \text{if } \bar{\lambda}_y \le 0.3 \text{ then } \mu_y \le 1.0 \text{ else } \mu_y \le 0.9 \end{gathered} \tag{19}$$

Finally, the equivalent uniform moment factor $\beta_{M,LT}$ can be determined in function of the bending diagram shape in the strong axis using expression (10), resulting in (for a mid span point load and for uniformly distributed load respectively, as in EC3 [16])

$$\beta_{M,LT} = \beta_{M,y} = 1.4 \text{ and } \beta_{M,LT} = \beta_{M,y} = 1.3 \tag{20}$$

3.4 Accuracy of the proposals

Again, direct comparisons with the numerical results are used to validate this new proposal. The graphics from Figure 3 were obtained for beam-columns with the possibility of occurring LTB, with bending in the strong axis. The length of 3 m corresponds to non-dimensional slenderness values of $\bar{\lambda}_z = 0.62$, while the length of 7 m corresponds to $\bar{\lambda}_z = 1.45$. The non-dimensional slenderness values for the LTB phenomena are given in the graphs. The interaction curves named "EN 1993-1-4 mod" in the graphs from Figure 6 were obtained from Part 1-4 of EC3 with the new proposal for columns [9] and with the new proposal for LTB [10].

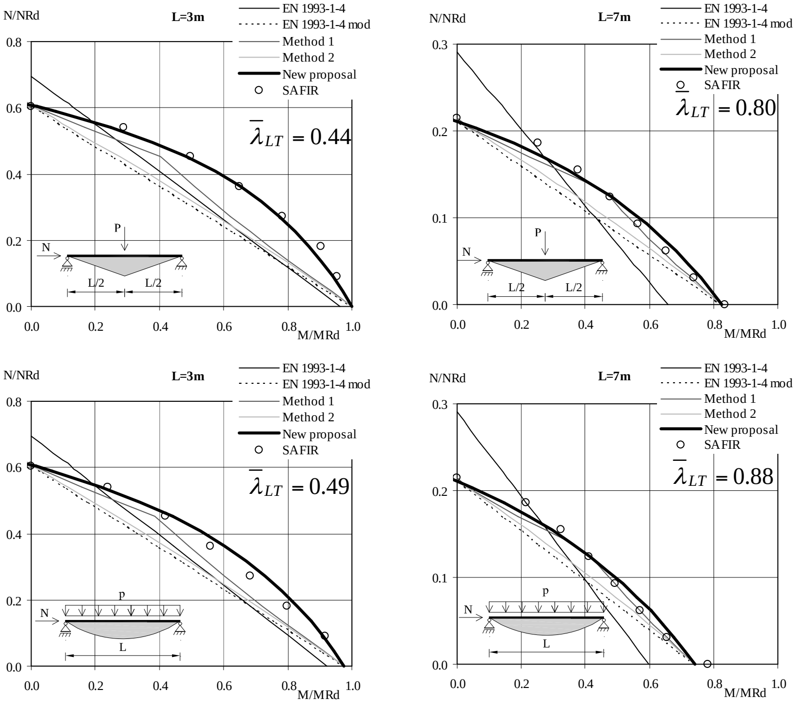

Figure 3: Comparison between different interaction curves for beam-columns with LTB.

Again, the method providing a better approximation to the numerical results is the “New proposal”. “Method 1” and “Method 2” also present safe approximations. From these two methods, the one that has a closer behaviour to the numerical results is “Method 1”.

It can also be observed that the new proposals, for columns [9] and for LTB of beams [10], introduce significant improvements in the interaction curves approximations when compared to the numerical results.

4 CONCLUSIONS

Different approaches for evaluating the safety of stainless steel elements subjected to axial compression and bending were presented and analysed. These approaches address the influence of global buckling phenomena (flexural buckling and LTB).

All the methods were tested with the new proposals, for columns [9] and for LTB [10], which introduced significant improvements in the interaction curves approximations when compared to the numerical results.

From the obtained results, it can be concluded that EC3 formulae for stainless steel beam-columns do not provide an accurate approximation to the real behaviour. Also, in order to use the new carbon steel interaction curves (Method 1 and Method 2) to stainless steel, additional modifications are needed.

The proposal that performed better is the one resulting from previous works by the authors [8], where new interaction curves, for the design of stainless steel beam-columns with and without LTB, were developed. This proposal has the advantage of being easier to use than Method 1 and Method 2. This paper only included two loading types (point load applied at the mid span and distributed load along the member length). However similar studies, on members with combined axial compression, end moments and transverse loads, have confirmed that it may be possible to use expression (10), from the ENV version of Part 1-1 of EC3 [6] and EN version of Part 1-2 of EC3 [16], for the calculation of the equivalent uniform moment factor β_M , since this proposal, follows this same approach and has shown to be accurate for other bending moment diagrams [8].

REFERENCES

[1] Estrada, I.; Real, E.; Mirambell, E., "General behaviour and effect of rigid and non–rigid end post in stainless steel plate girders loaded in shear. Part I: experimental study", *Journal of Constructional Steel Research*, Elsevier, Vol. 63, pp. 970–984, 2006.

[2] Gardner, L., "The use of stainless steel in structures", *Progress in Structural Engineering and Materials*, 7(2), pp. 45–55, 2005.

[3] Euro Inox and Steel Construction Institute, *Design Manual for Structural Stainless Steel*, 3rd edition, 2006.

[4] CEN, European Committee for Standardisation, *EN 1993-1-4, Eurocode 3: Design of steel Structures – Part 1-4: General rules – Supplementary Rules for Stainless steels*, Belgium, 2006.

[5] CEN, *EN 1993-1-1, Eurocode 3, Design of Steel Structures – Part 1-1: General rules and rules for buildings*, Belgium, 2005.

[6] CEN *ENV 1993-1-1 Eurocode 3: Design of Steel Structures - Part 1-1: General rules and rules for buildings*, Belgium, 1992.

[7] Boissonnade, N., Greiner, R., Jaspart, J. P., *Rules for Member Stability in EN1993-1-1 Background documentation and design guidelines*, ECCS TC 8 – Stability, 2006.

[8] Vila Real, P.; Lopes, N.; Simões da Silva, L. "Stainless steel beam-columns interaction curves with and without lateral torsional buckling", *proceedings of the ESMC2009 7th EUROMECH Solid Mechanics Conference*, Lisbon, Portugal, September of 2009.

[9] Lopes, N.; Vila Real, P.; Simões da Silva, L., "Numerical modelling of the Flexural buckling of axially loaded stainless steel members", *proceedings of the third International Conference on Steel and Composite Structures ICSCS07*, Manchester, United Kingdom, 2007.

[10] Lopes, N.; Vila Real, P.; Simões da Silva, L.; Mirambell, E., "Numerical modelling of the lateral-torsional buckling of Stainless steel I-beams: comparison with Eurocode 3", *proceedings of the Tenth International Conference on Enhancement and Promotion of Computational Methods in Engineering and science EPMESC X*, Sanya, China, 2006.

[11] Vila Real, P.; Lopes, N.; Simões da Silva, L.; Franssen, J.-M., "Lateral-torsional buckling of Stainless steel I-beams in case of fire", *Journal of Constructional Steel Res.*, ELSEVIER, 2008.

[12] Chen, W. F. and Lui, E. M., *Stability design of steel frames*, CRC Press, 1991.

[13] Gardner, L., Nethercot, D., "Numerical Modelling of Stainless Steel Structural Components - A consistent Approach", *Journal of Constr. Engineering*, ASCE, 2004.

[14] Franssen, J.-M., "SAFIR. A Thermal/Structural Program Modelling Structures under Fire", *Engineering Journal*, A.I.S.C., 2005.

[15] Talamona, D., *Flambement de poteaux métalliques sous charge excentrée, à haute température*, (in french); Ph.D. thesis, Université Blaise Pascal Ecole Doctorale Sciences pour l'Ingenieur de Clermont-Ferrand, 1995.

[16] CEN, *EN 1993-1-2, Eurocode 3, Design of Steel Structures – Part 1-2: General rules – Structural fire design*, Belgium, 2005.