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**A SIMPLE MODEL FOR THE LATERAL-TORSIONAL BUCKLING OF STEEL
I-BEAMS UNDER FIRE CONDITIONS: NUMERICAL AND
EXPERIMENTAL VALIDATION**

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ABSTRACT

When a beam is bent about its greatest flexural resistance axis it may twist and move laterally, before it reaches its strength limit. Although this problem of lateral-torsional buckling of steel beams at room temperature is well known, the same problem at elevated temperature is not.

This paper summarises the results obtained in the scope of a research project entitled "*lateral buckling of steel I-beams under fire conditions*", where a set of 120 experimental and numerical tests were performed on IPE 100 beams, submitted to temperatures varying from 20 °C to 600 °C, to validate a new proposal for the buckling resistance moment of a laterally unrestrained beam under fire conditions.

The numerical and experimental results for the lateral-torsional buckling of steel I-beams have been compared to those from the simple model presented in Eurocode 3, Part 1-2. This Eurocode simple model promotes safety levels that depend mainly on the non-dimensional slenderness of the beams. The analytical results provided by the Eurocode 3, for a certain range of the slenderness, would now appear to be unsafe when compared with the numerical and experimental results.

It will be shown that the new proposal is safer than the Eurocode 3 formulas.

1. INTRODUCTION

The behaviour of steel I-beams at elevated temperature has been analysed numerically (Vila Real & Franssen, 1999, 2001) being a new proposal made for the evaluation of its lateral-torsional buckling resistance.

This new proposal contains a scalar β , which has to be calibrated to ensure an appropriate safety level, which is done in this work throughout a large set of experimental tests.

Although the problem of lateral-torsional buckling of steel I-beams at room temperature is well known (Timoshenko, 1963 & Trahair, 1993, 1998 & Papangelis, 1998) the same problem at elevated temperature is not. Among the work done in this field there is the paper from Bayley et al., 1996, who uses a three dimensional computer model to investigate the ultimate behaviour of uniformly heated unrestrained beams. The proposal the present paper pretends to validate was based on the numerical results from the SAFIR program, a geometrical and materially non-linear code specially established for the analysis of structures submitted to fire (Franssen, 1995). The capability of this code to model the lateral-torsional buckling of beams has been shown (Vila Real & Franssen, 1999) at room temperature by comparisons against the formulas of the Eurocode 3, Part 1-1 (1993) and by Franssen (1994) who have compared the SAFIR program with other four structural codes.

It must be emphasized that the simple model that this paper wants to validate, presented by Vila Real et al. (1999,2001), was based on numerical simulations using characteristic values for initial out-of-straightness ($L/1000$) and residual stresses ($0.3 \times 235 \text{ MPa}$), which are unlikely to be simultaneously present in a test or in a real fire. In this work the geometrical imperfections and the residual stresses were measured as well as the nominal yield strength of the material and the Young Modulus. These measured values were used in the numerical calculations.

A set of 120 full-scale tests based on a reaction frame and on a hydraulic system has been carried out for beams of the European series IPE 100 with lengths varying from 0.5 to 6.5 meters. Three tests have been done for each beam length and for each temperature level. The beams were electrically heated by means of ceramic mat elements, heated by a power unit of 70 kVA. To increasing the thermal efficiency a ceramic fibre mat has been used to improve the thermal insulation.

2. EXPERIMENTAL AND NUMERICAL CASE STUDY

The simple supported beam with fork supports shown in figure 1 has been studied.

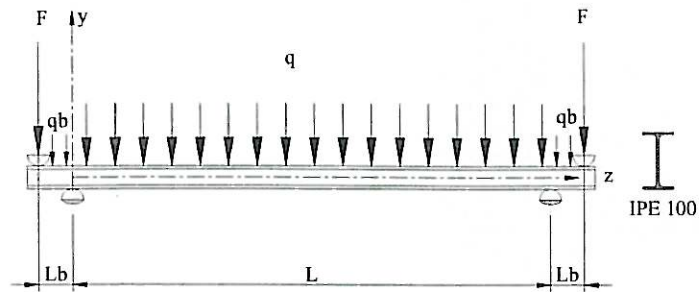
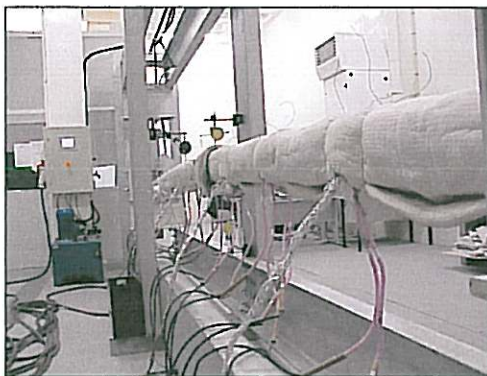


Figure 1: Case study. Simple supported beam with forks supports.

In this figure, q_b represents the self weight of the beam and q represents the additional distributed load due to the weight of the ceramic mat and electro-ceramic resistances.

The experimental set-up is shown in figure 2, where the fork supports, the hydraulic jacks and the ceramic mat elements can be seen. Automatic control of separated heating elements was used in order to ensure a uniform temperature distribution along the length of the beams. The temperature was measured with thermocouples welded on the beams at different locations along the beam length.



a)



b)

Figure 2: a) Experimental setup. b) Fork of support and hydraulic jack.

Three types of mid span displacements were experimentally measured. The thermal action was changed from room temperature up to 200, 300, 400, 500 and 600 °C. These temperatures were applied before the mechanical action, with the longitudinal displacement unrestrained and kept constant during the loading, which is applied only after the temperature stabilisation.

The stress strain relationship used is a function of the measured material strength and varies with temperature, according to Eurocode 3, Part 1-2 (1995).

A three-dimensional beam element with 15 degrees of freedom and three nodes has been used to numerically calculate the buckling moment resistance of the beams loaded as shown in figure 1. All the measured geometrical, mechanical properties and residual stresses were used in the numerical calculation.

3. THE NEW PROPOSAL FOR LATERAL-TORSIONAL BUCKLING RESISTANCE

A new proposal for the lateral-torsional buckling resistance, based on numerical calculations was proposed by Vila Real et al (1999,2001). According to this new proposal, and adopting for the lateral-torsional buckling the same proposal from Franssen et al. (1995) to represent the behaviour of axially-loaded columns when submitted to fire conditions, the design buckling resistance moment of a laterally unrestrained beam with a class 1 or 2 cross section type, can be calculated by

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

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

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

with

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

The imperfection factor α , in this proposal, is a function of a severity factor β

$$\alpha = \beta \varepsilon \quad (4)$$

The severity factor β , which should be chosen in order to ensure an appropriate safety level, has been taken as 0.65 (Vila Real & Franssen, 1999, 2001), and the material factor ε is given by

$$\varepsilon = \sqrt{\frac{235}{f_y}} \quad (5)$$

where f_y represents the nominal yield strength of the material in MPa.

The remaining factors should be calculated as in the Eurocode 3.

Comparing this new proposal with the Eurocode 3 formulas it can be verified that the shape of the buckling curve is different, with the new starting from $\chi_{LT} = 1.0$ for $\bar{\lambda}_{LT} = 0.0$ but decreasing even for very low slenderness, instead of having a horizontal plateau up to $\bar{\lambda}_{LT} = 0.4$ as in the present version of the Eurocode 3 (1995).

The lateral-torsional buckling curve now depends on the steel grade due to the parameter ε that appears in the imperfection factor as it can be seen in figure 3. In this figure on the vertical axis it is plotted the ratio

$$\frac{M_{b,fi,t,Rd}}{M_{fi,\theta,Rd}} = \chi_{LT,fi} \quad (6)$$

where, $M_{b,\bar{f},t,Rd}$ is the design buckling resistance moment at time t of a laterally unrestrained beam given by equation (1) and $M_{\bar{f},0,Rd}$ is the design moment resistance of a Class 1 or 2 cross-section with a uniform temperature θ_a given by

$$M_{\bar{f},0,Rd} = k_{y,0} \frac{\gamma_{M0}}{\gamma_{M,\bar{f}}} M_{Rd} \quad (7)$$

where, $\gamma_{M0} = 1.0$, $\gamma_{M,\bar{f}} = 1.0$ and M_{Rd} is the plastic resistance of the gross cross-section $M_{pl,Rd}$ for normal temperature, which is given, using $\gamma_{M0} = 1.0$, by

$$M_{Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} \quad (8)$$

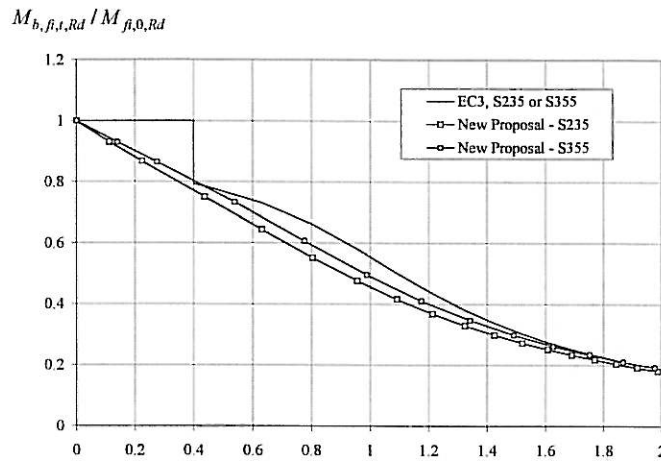


Figure 3: Comparison between design buckling curve and the new proposal.

4. CRITICAL MOMENT

The critical moment, M_{cr} , necessary to evaluate the non-dimensional slenderness $\bar{\lambda}_{LT,0,com}$, according the Eurocode 3 is obtained solving the following differential equations

$$\begin{aligned} (EI_y u''')'' + (M_x \phi)'' &= 0 \\ (EI_w \phi'')'' - (GI_t \phi')' + (M_x u'') &= 0 \end{aligned} \quad (9)$$

which traduce the lateral-torsional buckling equilibrium of the beam. The first equation represents the equality at equilibrium between the out-of-plane bending action $-(M_x \phi)''$ and the flexural resistance $(EI_y u''')''$ and the second equation represents the equality between the torsion action $-M_x u''$, and the warping and torsional resistances $(EI_w \phi'')''$ and $-(GI_t \phi')'$. The bending moment

distribution M_x due to the transverse load q varies along the beam and so the differential equations have some variable coefficients and are difficult to solve (Trahair, 1993).

It can be verified that these differential equations are satisfied by the substitution of the buckled shapes formulas:

$$\frac{u}{\delta} = \frac{\phi}{\theta} = \frac{z}{L} - \frac{z^2}{L^2} \quad (10)$$

where δ and θ represent the values of u and ϕ at mid span and z represents the co-ordinate along the beam axis.

Substituting the equation (10) and all the derivatives into the energy equation (11) and taking into account the moment distribution along the buckling length due to the uniformly distributed load, it can be verified that the critical load F (see Fig. 1) is a function of the material properties, the beam cross section geometric characteristics and also a function of the distributed load.

$$\frac{1}{2} \int_0^L (EI_y u''^2 + EI_w \phi''^2 + GI_t \phi'^2) dz + \frac{1}{2} \int_0^L 2M_x \phi u'' dz + \frac{1}{2} \int_0^L q(y_q - y_0) \phi^2 dz = 0 \quad (11)$$

This critical force F when introduced into the moment distribution, gives at the supports the critical moment, M_{cr} . This moment can be compared to the critical elastic moment, M_{cr}^{pb} for the pure bending case using the moment distribution factor α_M (Trahair, 1993 Trahair & Bradford, 1998) as shown in the following equation

$$M_{cr} = \alpha_M M_{cr}^{pb} = \alpha_M \frac{\pi^2 EI_x}{(kL)^2} \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_x} + \frac{(kL)^2 GI_t}{\pi^2 EI_x}} \quad (12)$$

where k represents the effective beam length factor and k_w the factor which accounts for the beam end warping. Regarding the type of load and support conditions used in the experimental tests, the value of $k = 0.5$ should be used to represent total constraint of the lateral movement (see figure 4) and the value of $k_w = 1$ to the free end warping condition.

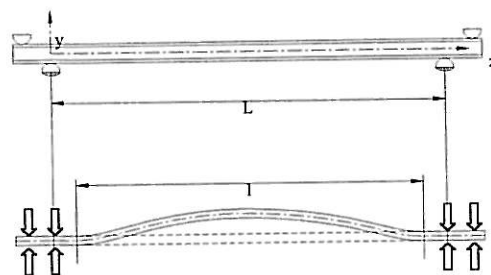


Figure 4: Effective buckling length, $l = kL$.

The analytical calculation showed that the moment distribution factor α_M is not constant and depends on the buckling length of the tested beam as shown in figure 5.



Figure 5: Moment distribution factor α_M .

5. EXPERIMENTAL EVALUATION

A multifunction reaction structure shown in figure 2 was used to test the beams at elevated temperatures and apply the mechanical loads. The loads were applied by means of two hydraulic jacks with 60 ton each and the beams were heated using electric ceramic mats.

It was used 500 meters of IPE 100 profile, given 120 testing beams with lengths varying from 0.5 to 6.5 meters.

The initial conditions of the steel beams were measured, specially, the residual stresses, the geometric imperfections and the cross section geometry was dimensionally controlled.

The residual stresses were measured in four points of the cross sections throughout the drill hole method.

Two types of geometric imperfections were measured. One related to the cross section dimensions, measured by digital callipers and the second one related to the longitudinal lateral distance relatively to a imaginary straight line, measured by a laser beam method.

A set of 31 profiles from the 46 originals was used to compare the cross section strength related to geometric data. The calculated plastic modulus exceeds the foreseen values.

A set of 20 specimens was tested on the 4485 Instron universal machine with 200 kN maximum capacity. The specimens were machined from the flanges and web parts from the IPE100 beams, and follow the Portuguese standard NP EN10002-1 (1990), for yield strength and elastic modulus characterisation.

Two different types of electro ceramic mat resistances with 1220 x 45 and 610 x 85 mm, with a maximum electric power of 2.7 kW each, were used to heat the beams. This material is capable to support temperatures up to 1050 °C, although our experiments were done only up to 600 °C and with a heat rate of 800 °C/h.

The experimental results correspondent to all the tested temperatures were plotted in the same chart as shown in figure 6. It can be seen that the curve of the new proposal with $\beta = 0.65$ is safer than the curve of the Eurocode 3.

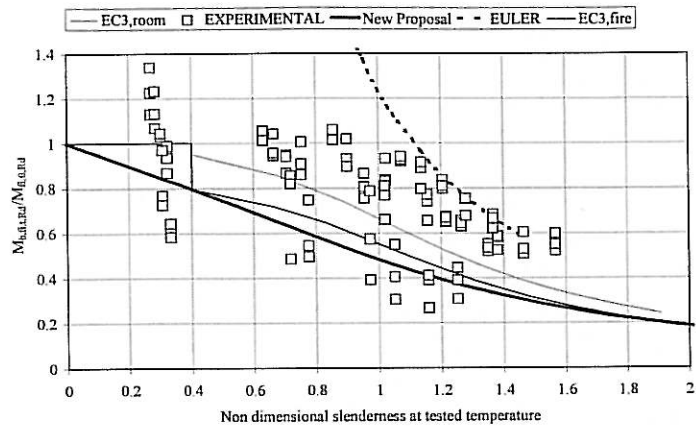


Figure 6: Beam design curves at elevated temperatures. Experimental results.

6. NUMERICAL EVALUATION

A set of 120 numerical calculations was made to calculate the buckling resistance moment at elevated temperatures. A non-linear material and geometrical code, based on two types of finite elements, made possible the study of lateral-torsional buckling of the IPE 100 beams. Bi-dimensional plane linear elements were used to describe the temperature result from the thermal action. The warping function and the torsion resistance have been calculated for each temperature level, considering the beams geometric imperfection, the residual stresses and the material property values, according to the experimental measuring and their temperature dependence according to the Eurocode 3.

The buckling resistance moment obtained by numerical simulation were plotted in the chart of figure 7. These numerical results show once more that the new proposal is safer than the eurocode 3 curve.

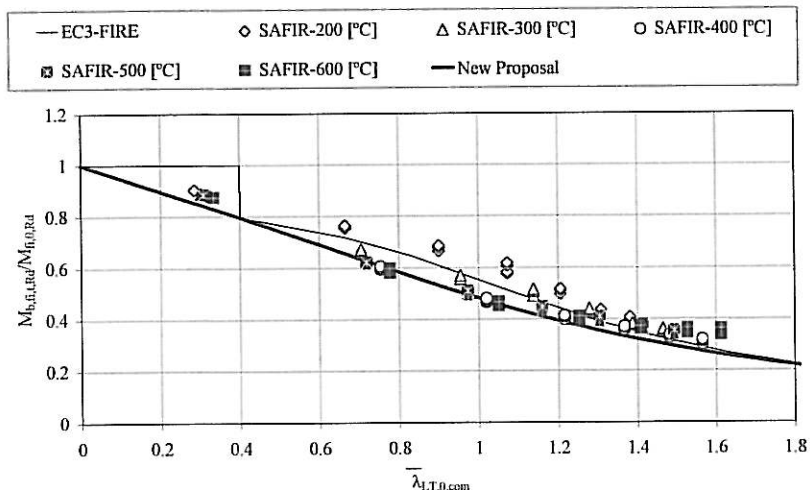


Figure 7: Beam design curves at elevated temperatures. Numerical results.

7 EXPERIMENTAL AND NUMERICAL COMPARISONS

Both experimental and numerical results have been compared with the simple formulas from Eurocode 3 Part 1.2 and the new proposal. The results of these comparisons are shown respectively in figure 8 and figure 9.

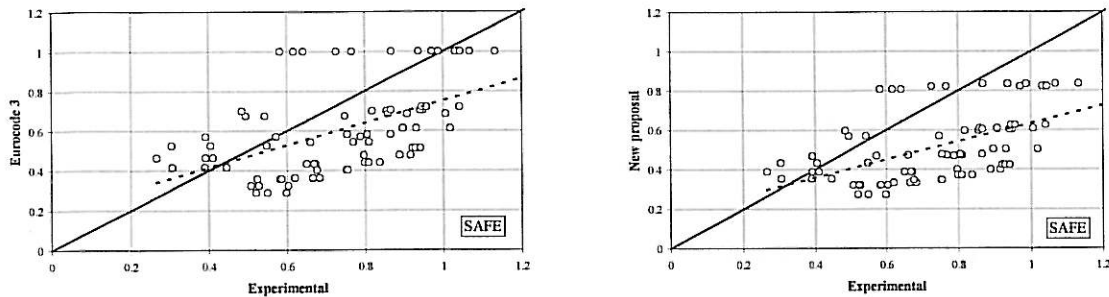


Figure 8: Experimental behaviour, for higher temperatures (above 400 °C).

The regression line is much more close to the ideal continuous line in the case of numerical calculation than for the experimental results but in both cases the number of unsafe points is smaller for the new proposal. It is clear from this figures that the new proposal with $\beta = 0.5$ is safer than the Eurocode 3.

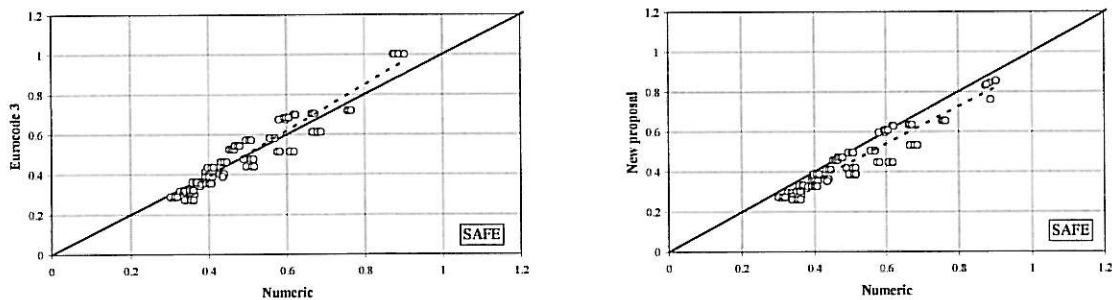


Figure 9: Numerical behaviour, for higher temperatures (above 400 °C).

8. CONCLUSIONS

The physical fact that elasticity modulus decreases faster than the yield strength when the temperature increases, plus the fact that the stress-strain relationship at elevated temperature is not the same as at room temperature, produce a modification of the lateral-torsional buckling curve at elevated temperature. The horizontal plateau valid at 20 °C up to a non-dimensional slenderness of 0.4 vanishes at elevated temperatures. The severity factor $\beta = 0.65$ earlier suggested in a previous work from the authors (Vila Real & Franssen, 1999, 2001) has confirmed and it was shown that the new proposal for lateral-torsional buckling, is safer than the Eurocode 3 formulas (1995).

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