# STRUCTURES IN FIRE

# PROCEEDINGS OF THE FIRST INTERNATIONAL WORKSHOP

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Workshop web page on http://www.ulg.ac.be/spec

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# FIRST INTERNATIONAL WORKSHOP - STRUCTURES IN FIRE

COPENHAGEN, the 19th and 20th of June, 2000

## *Co-organised by*

University of Liege,
Danish Institute of Fire Technology,
CIB-W14 Fire.

## *Responsible for*

the local organising committee:
*Henrik Bygbjerg*,
Danish Institute of Fire Technology

the concept, the proceedings:
*Jean-Marc Franssen*,
University of Liege

the program:
*Alexander Ptchelintsev*,
Technical Research Centre of Finland.

## *Sponsor*

Danish Institute of Fire Technology

# Why this workshop?

Looking back some 10 or 20 years ago, the scientific community dealing with the problem of fire in buildings could, schematically, be divided into two separate groups; one group was dealing with the fire side of the problem and was considering that the temperature of 540°C was anything which had to be known concerning the structure of the building; the other group was investigating the behaviour of the structure, quite happy with the comfortable feeling that the ISO curve was a perfect representation of the fire.

The disadvantages of this situation with very little, if any, communication between the two groups progressively became more and more evident and people started to consider talking to each other and, even more, to widen their field of investigation and have a look into the other guy's garden. Other aspects also came into consideration such the behaviour of human beings, risk analysis, etc. This evolution lead to the now widely accepted concept of Fire Safety Engineering which, simply saying, is nothing more than the fact that we are starting to treat the problem of Fire Safety in the way that engineers treat other problems, i.e. trying to do their best in order to take into consideration every phenomenon which is suspected to play a role.

There was anyway a positive aspect to the situation prevailing in these old days: it was very easy and common to meet and discuss with the few people who had a real expertise in your field of application. Specialised meetings were regularly organised in which all the experts who counted would normally show up. For the structural analysis, some examples are:

the ECCS workshop on material properties at elevated temperatures, by ECCS committee 3 - Fire safety of steel structures, in Arnhem, The Netherlands, in June 1986,

the EGOLF seminar "Protection contre l'incendie des structures en acier. Harmonisation Européenne", in Brussels, Belgium, in November 1986,

the Abschlusskolloquium "Bauwerke unter Brandeinwerkung", Technische Universität Braunschweig, in Braunschweig, Germany, in April 1987,

the Eurocodes, Structural Fire Design, Seminar organised by the Eurocode fire drafting groups, in Luxembourg, in June 1990,

the 3rd CIB/W14 Fire Safety Engineering Workshop on Modelling, in Rijswijk, The Nederlands, in January 1993,

or, to some extend, the First European Symposium on Fire Safety Science, IAFSS, in Zürich, Switzerland, in August 1995.

The field of interest was certainly too narrow, but the progress were spectacular. Nowadays, the same specialists and their presentations tend to be disseminated in different places and various meetings: for the organiser of every structural conference on steel, on concrete or on wood, this looks much smarter if he has a session on such an exotic topic as fire, and a couple of papers are indeed published in these general conferences but the interest of the public is generally poor, and very few of those who would really be interested are present. The same problem holds for the publications that, as academic, many researchers have to present in some journals which are prestigious but have only a marginal interest for fire.
Concerning the big conferences specifically dedicated to the fire – and leaving apart that some of them appear now to be concurrent which is another reason of dissemination – if they are of the highest importance because they allow to open your eyes to other aspects than those that you treat in your everyday life, it has to be recognised that the number of presentation is so high that very short time can be dedicated to discussion and that it is not easy to have detailed information on a specific topic.

At the end of 1998 and during the year of 1999, the idea was circulating that a specific association could be created on the topic of structural fire modelling. An exchange of e-mail messages followed and, finally, the topic was discussed in July at an informal meeting in Poitiers during the IAFSS symposium. The general opinion was that it would be better not to have a new association, because there are already so many of them. In order to promote a more intense circulation of information among those interested by the subject, two actions were decided: one was the creation of the SiF discussion list on internet, and the second one was the organisation of this workshop.

## PROGRAM AND LIST OF CONTENT

**Monday 19.06.2000**

| | | |
|---|---|---|
| 9:00 – 9:30 | **Opening of the Workshop** | |

### Session 1

### Session 2

### Session 3

## Session 4

| | | |
|---|---|---|
| 15:10 – 15:35 | **A. Ptchelintsev**<br>*A Comparative Thermal Analysis of Structures Exposed to Fire with Advanced Calculation Models* | **149-157** |
| 15:35 –16:00 | **L. Twilt, P.H.E. v.d. Leur & C. Both**<br>*Characteristics of the Heat Transfer for Calculating the Temperature Development in Structural Steelwork Exposed to Standard Fire Conditions under Plate Thermocouple Control* | **159-171** |
| 16:00 – 16:25 | **M. Green**<br>*The recent U.K. BS9999 – Presentation and worked examples* | **179** |

18:00 Technical visit of the ***Tivoli*** amusement park in the center of Copenhagen

**Tuesday 20.06.2000**

## Session 5

| | | |
|---|---|---|
| 9:00 –9:25 | **Y. C. Wang & J. M. Davies**<br>*Design of thin-walled steel channel columns in fire using Eurocode 3 Part 1.3* | **181-193** |
| 9:25- 9:50 | **D. Talamona & J.-M. Franssen**<br>*New quadrangular shell element in SAFIR* | **195-210** |
| 9:50- 10:15 | **J. Myllymäki & M. Kokkala**<br>*Thermal Exposure to a High Welded I-Beam Above a Pool Fire* | **211-224** |
| 10:15 –10:40 | **B. Zhao & D. Joyeux**<br>*Evaluation of fire resistance of open car park under natural fire condition with advanced calculation models* | **225** |
| 10:40 – 11:00 | **Coffee Break** | |

## Session 6

| | | |
|---|---|---|
| 11:00 – 11:25 | **G. Faller**<br>Towards a *Performance Based Design Approach for Fire Resistance Grading of Buildings* | **227-241** |
| 11:25 – 11:50 | **W.E. Koffel**<br>*With Performance Codes Who Needs Structural Fire Resistance?* | **243** |
| 11:50 – 12:15 | **M. Gillie & A.S. Usmani**<br>*An Analysis of the Behaviour of the First Cardington Test Using Stress-Resultant Shell Elements* | **245-266** |
| 12:15 – 12:40 | **J. Outinen, O. Kaitila & P. Mäkeläinen**<br>*A Study for the Development of the Design of Steel Structures in Fire Conditions* | **267-281** |
| 12:40 – 13:30 | **Luncheon** | |

## Session 7

| | | |
|---|---|---|
| 13:30 – 13:55 | **K. D. Hertz**<br>*A Survey of a System of Methods for Fire Safety Design of Traditional Concrete Constructions* | **283-292** |
| 13:55 – 14:20 | **N. E. Andersen**<br>*Calculation and Testing of Factory-made Concrete Elements* | **293-303** |
| 14:20 – 14:45 | **S. Attia**<br>*Investigation in Eurocode for concrete columns* | **305-322** |
| 14:45 – 15:10 | **J.-M. Franssen**<br>*Design of concrete Columns Based on EC2 Tabulated Data – A Critical Review* | **323-340** |
| 15:10 – 15:40 | **General Discussion & Closing of the Workshop** | |

# Lateral Torsional Buckling of Steel I-Beams in Case of Fire – Experimental Evaluation

**Piloto, P. A. G.[1]; Vila Real, P. M. M.[2]**

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## *ABTSRACT*

In this paper, experimental tests on lateral torsional buckling of steel I Beams under fire conditions are presented. The beam spans varies from 0.5 to 6.5 [m] length and the temperature was risen up to 600 [°C]. The initial conditions of the steel beams were measured. The residual stress state was characterized by the hole drilling method, using specific strain gauges. The geometric imperfections was measured by means of a laser beam and the cross section of the beams was dimensionally controlled. The methodology to the thermo-mechanical load was first heat the beam and let it expand for controlling the mechanical load position, and finally increase the load step by step.

The aim of this work is to validate the proposal for lateral torsional buckling design resistance suggested in ref.[1] based on numerical results.

A Set of experimental results are presented, relating the collapse load with the mid span movement of the beam cross section, when submitted to concentrated moments at the ends and to a uniform distributed load, due to the ceramic mat, the insulation material weight and the self weight of the beams.

## 1 - INTRODUCTION

The lateral torsional buckling resistance of steel beams is well known at room temperature, but in case of fire, the guides for designers are undifferentiated regarding the temperature and they are not supported by experimental results. In this work it is presented a full scale test at elevated temperatures for determining the buckling design resistance of simple supported steel I beams.

Some numerical simulations of the same test are being made and should be presented soon. The behavior will be material and geometrical non linear.

The tests presented were done as a result of a Portuguese R&D national project PRAXIS/P/ECM/14176/1998 "lateral buckling of steel beams under fire conditions" and intend to be a contribution on the knowledge of structures in fire.

The Experimental set-up is presented in the figure 1 and is constituted by two parts. One for the thermal effect simulating of the fire around the beam, and the other for structural purpose.

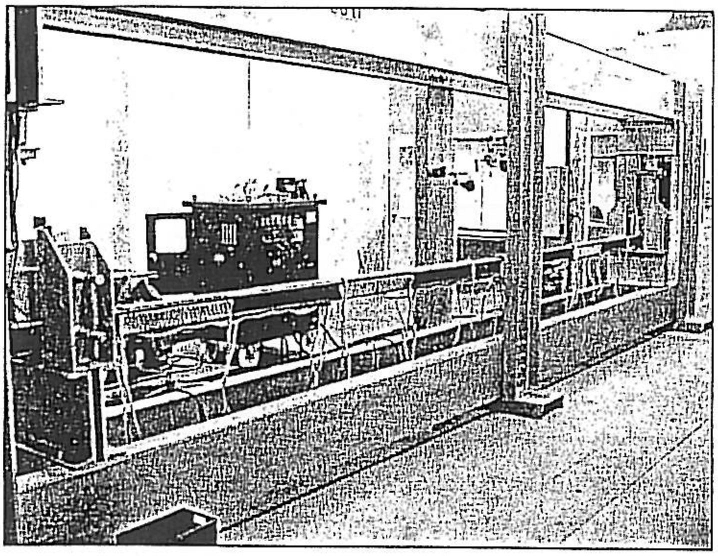


**Fig.1 – Experimental set-up for lateral torsional behavior of structures.**

The heating system must have the necessary components for thermal energy generation. The temperature controller connected to several thermocouples along the beam should be able for rise and fall the temperature, maintaining the temperature as uniform as possible. The heating elements should deliver the necessary power, provided the thermal insulation for best efficiency and a set of accessories for mounting the complete system are required.

The structural system should be stable with adjustable supports and load points. The structure used is modular and multi- functional. The Electro Hydraulic system is capable of delivery 60 [ton] force in each point load, and has the possibility of programming the rise and rate of force respect to time. The control unit as the capability of store the pick force value.

According to the Eurocode 3, the design buckling resistance moment of a laterally unrestrained beam with class 1 or 2 cross section, in case of fire is given by

$$M_{b,fi,t,Rd} = \frac{\chi_{LT,fi}}{1.2} w_{pl,y} k_{y,\theta,com} f_y \frac{1}{\gamma_{M,fi}} \tag{1}$$

Provided that the non dimensional slenderness $\overline{\lambda}_{LT,\theta,com}$ for the maximum temperature in the compression flange $\theta_{a,com}$ reached at time t does not exceed 0.4 no allowance need be made for this situation. When non dimensional slenderness exceed that value the design moment should be calculated by expression (1). In this expression $\chi_{LT,fi}$ represents the reduction coefficient in fire situation, $w_{pl,y}$ is the plastic moment of beam cross section, $k_{y,\theta,com}$ is the reduction factor for the influence in yield by the temperature variation.

The aim of this work is to contribute to an alternative expression for the design moment resistance and validate the numerical results from [1] with full scale tests.

## 2 - LATERAL TORSIONAL BUCKLING OF STEEL I BEAMS

When a beam is bent about its greatest flexural axis of rigidity it may twist before it reaches its strength limit state. This stability limit state is most commonly referred to as *lateral torsional buckling* of a beam. The twisting of the beam occurs when the compression flange becomes unstable as a result of its being subjected to flexural induced axial stresses. Lateral buckling is of importance when the compression flange is laterally unsupported as is often the case in continuous beams, cantilever beams, frame beams and frame columns.

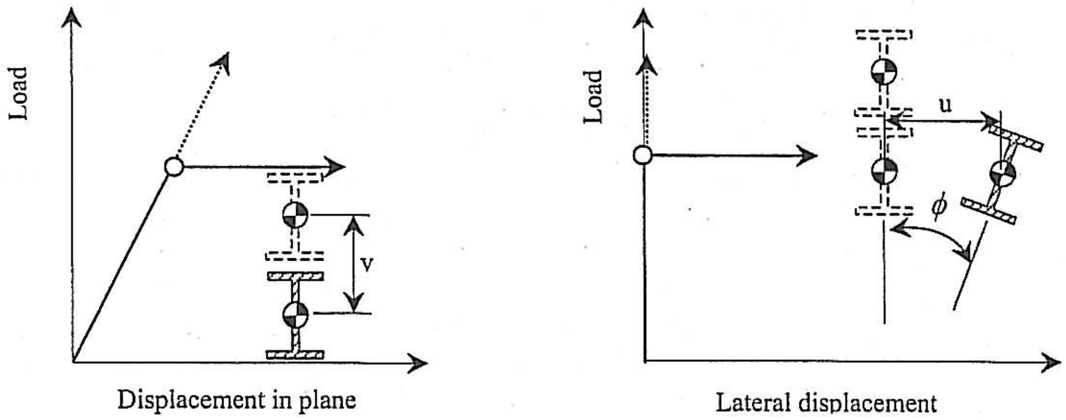


**Fig.2- Graphics representation of the cross section movement.**

The lateral torsional buckling of beams (Figure 2) involves lateral displacement u out of the plane of bending and twist rotations $\phi$. In this case, the twist rotations make the applied moments to have components acting out of the original plane of bending, while the lateral rotations $du/dz$ ($z$ is the coordinate along the beam axis) cause the applied moments to have torque components about the axis of twist through the shear center.

Methods for designing against lateral torsional buckling are essential of two types. For the first type, buckling is avoided, and the member in plane capacity is fully utilized. One way of achieving this is to use beam cross sections not susceptible to buckle. A second way of avoiding buckling is to increase bracing, either by reducing its spacing, or else by increasing its effectiveness. For the second type a reduced capacity is determined which accounts for the effects of flexural torsional buckling [3].

In this paper a simple supported beam with two forks at the supports, uniform distributed load (due to the weight of the heating system) and moment at the ends of the beam, as shown in the figure 3, is studied.

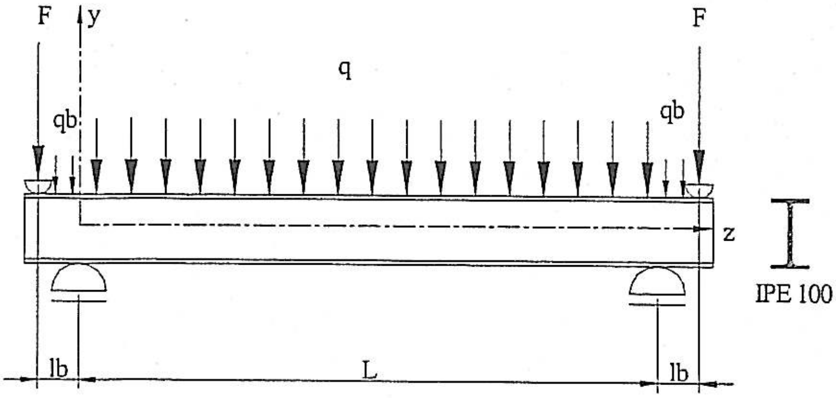


**Fig. 3 – Case studied. Simply supported beam with two forks separated by L[m].**

The bending moment distribution with transverse load varies along the beam and so the differential equations have some variable coefficients and are difficult to solve.

For the case when the load acts at the shear center, and for double symmetric beams, the elastic critical moment varies with the type of load.

The lateral torsional buckling equilibrium is traduced by the differential equations presented in (2).

$$\begin{gathered}\left(EI_y u''\right)'' + \left(M_x \phi\right)'' = 0 \\ \left(EI_w \phi''\right)'' - \left(GJ\phi'\right)' + \left(M_x u''\right) = 0\end{gathered} \tag{2}$$

The first equation expresses the equality between the flexural resistance $\left(EI_y u''\right)''$ and the lateral bending action $-\left(M_x \phi\right)''$ of the bending moment caused by this rotation. The second equation expresses the equality between the sum of internal warping and uniform torsion resistance $\left[\left(EI_w \phi''\right)'' - \left(GJ\phi'\right)'\right]$ and the distributed torque generated by warping and twisting of the beam, during buckling.

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

$$\frac{u}{\delta} = \frac{\phi}{\theta} = sin\left(\frac{\pi z}{L}\right) \tag{3}$$

or still by the simply formula

$$\frac{u}{\delta}=\frac{\phi}{\theta}=\frac{z}{L}-\frac{z^2}{L^2} \tag{4}$$

where $\delta$ and $\theta$ represent the values of $u$ and $\phi$ at mid span and z the coordinate along the beam axis.

For the present case the beam should verify the equilibrium equations (2) and also the energy equation (5).

$$\frac{1}{2}\int_0^L\left(EI_y u''^2+EI_w\phi''^2+GJ\phi'^2\right)dz+\frac{1}{2}\int_0^L 2M_x\phi\, u'' dz+\frac{1}{2}\int_0^L q(y_q-y_0)\phi^2\, dz=0 \tag{5}$$

which represents the equality at buckling between the flexural, warping and torsional strain energy stored and the work done by the bending moment $M_x$ and the distributed load q, acting at a distance $y_q$ from the shear center $y_0$.

Substituting the equation (4) and all the derivatives into equation (5) and taking into account the moment distribution along the buckling length, it can be verified that the critical load is a function of the material properties, the geometric characteristics of the beam cross section and also a function of the distributed load. This critical force when introduced into the moment distribution, near one of the supports give the critical moment. That result can be compared to the critical elastic moment for the constant moment load case using the buckling factor $\alpha_M$, as shown in equation (6).

$$M_{cr}=\alpha_M\times\frac{\pi^2 EI_z}{L^2}\times\sqrt{\frac{I_w}{I_z}+L^2\times\frac{GI_t}{\pi^2 EI_z}} \tag{6}$$

This coefficient is not constant and depends on the buckling length of the tested beam, as can be seen in the figure 4.

The critical moment is necessary for the evaluation of the relative slenderness $\bar{\lambda}_{LT}$ in equation (10).

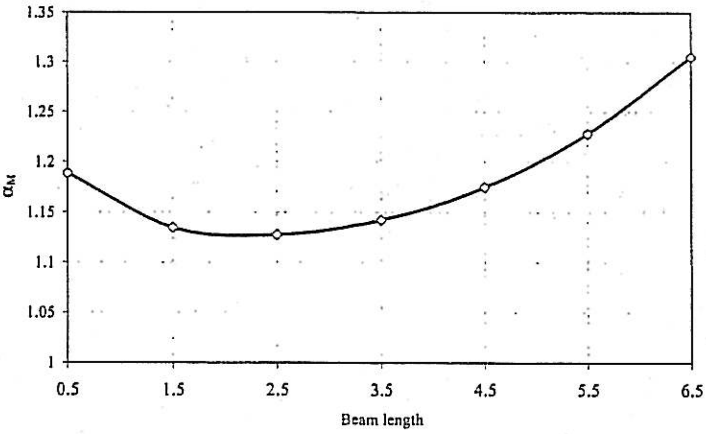


**Fig.4- Coefficient for critical elastic moment.**

## 3 - EXPERIMENTAL SETUP FOR EVALUATE LATERAL TORSIONAL BUCKLING

As a result of the R&D project it was necessary to build a support structure and all the necessary equipment for loading and measuring the necessary parameter during this phenomenon (figure 1). A multifunction structure with 8 x 1.2 [m] was used to fixe the beam and apply the forces. The Electro hydraulic power system with two hydraulic jacks with 60 [ton] each gave the possibility to simulate the mechanical action on the beams and the electric ceramic mat were used to simulate a fire condition, rising and controlling the temperature in the way we intended to do.

The initial conditions of the steel beams were measured, specially, the residual stresses, the geometric imperfections and the cross section geometry was dimensionally controlled. Most of this information was a result of the steel process fabrication and of the packing process during transportation and storing.

The rolling process reduces the thickness of the section and changes its shape. After the rolling phase, the steel will gradually cools. The cross sections will have a non uniform temperature distribution and the root of the web maintains its bigger temperature for a long period than the other parts. This differential cooling leads to residual stresses that can influence the behavior of steel work under load.

The residual stresses in a single structural component or in a global structure are always present even without service load. Fabrication processes like foundry, soldering, machining, heat treatment and other factors, are the most common causes in this stress state. Other possible causes are those related to structural repair or modifications in their components. In some cases the stresses can be introduced in the structure by means of installation procedures, over load or other type of variable loads.

The effects of residual stresses in structural components may be positive or negative, depending on the magnitude, signal and their distribution relative to those induced by external loads. Several reported cases presents these residual states as the predominant factor for structural collapse.

### 3.1 - Auxiliary equipment for experimental setup

A multifunctional and dimensional stable structure was built for the experimental tests of lateral torsional buckling of I beam under fire condition. This structure has two main types of UNP profiles and also HEA200 profiles to build the two movable forks supports. The two other movable point loads (see figure 1) are constituted by parts of UNP350. This flexibility is necessary to leave the beam expands during the fire simulation, and load the structure after fixing the supports

For fire simulation, a heating system with 70 [kVA] and all the necessary components for thermal energy generation were used. The temperature control for rise and fall should be done, the heating elements should deliver the necessary power, provide the thermal insulation for best efficiency.

Two different types of Electro ceramic mat resistance's with 1220 x 45 and 610 x 85 [mm] with the maximum electric power of 2.7 [kW] each were used for thermal delivery into steel I beams. This material is capable to support 1050 [°C], although our experiments were done up to 600 [°C] and at a heat rate of 800 [°C/h]. The temperature distribution along the beam should be uniform to compare with the numerical simulation. Although there is always a difference near the extremes of the tested beams as it can be proved by the registration of the temperatures of the thermocouples K type used.

The displacements of the three point controlling the cross sections movement were measured by means of displacement transducers as shown in the figure 5.

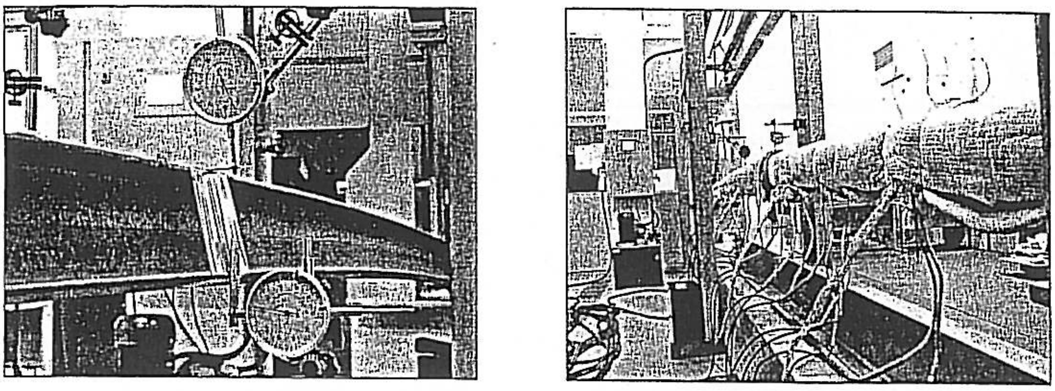


**Fig.5- Displacement measuring system at room temperatures and at elevated temperatures.**

### 3.2 - Methodology of the experiments

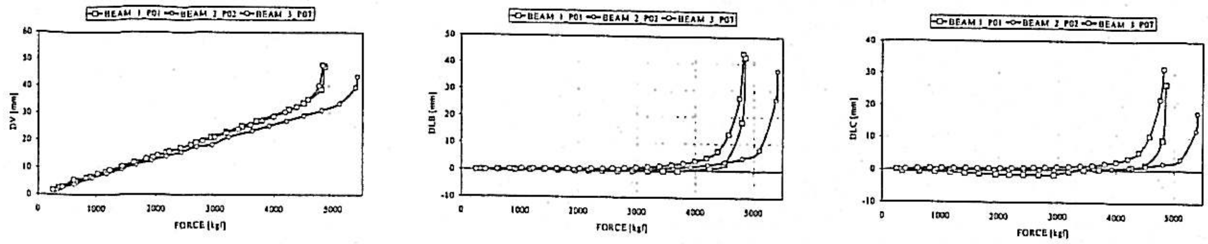
The geometric imperfections were measured for each I beam. For the beams submitted to elevated temperatures it was necessary to thermocouples to measure the temperature. This thermocouples were micro welded by a special equipment that minimize some possible reading errors with this type of thermal sensor.

During temperature rise the distance between the support and the point load was controlled and fixed after temperature stability was achieved. After that, the mechanical load is applied and incremented up to the collapse load, as can be seen in the figure 6.

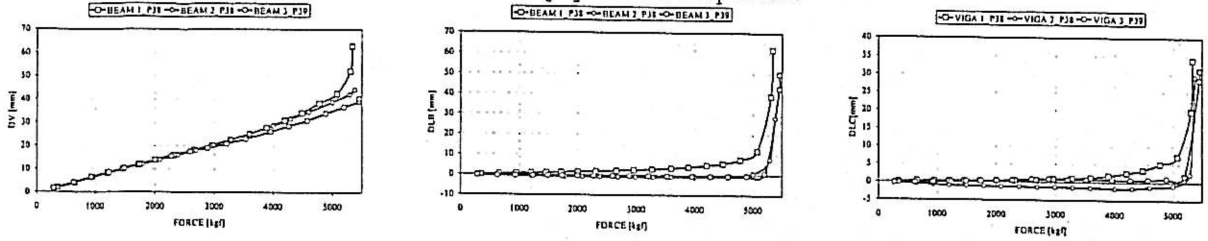
From the results presented in the figure 6 it can be seen that the collapse load decrease as the temperature of fire increase. The relation between displacement and force changes due to the variation of the material properties, as it can be shown in the graphics for temperatures above 400 [°C].

A series of results for other beam lengths were obtained and transposed to the global graphic results presented in figure 7.

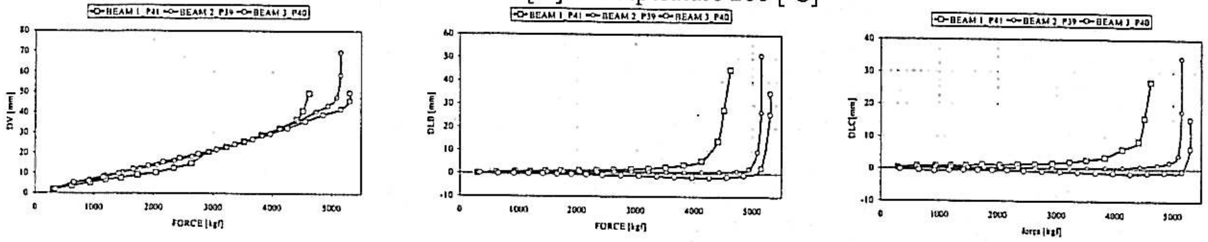
In all this graphic representation DV represents the vertical displacement, DLB represents the bottom lateral displacement and DLC the lateral top displacement of the mid span cross section.



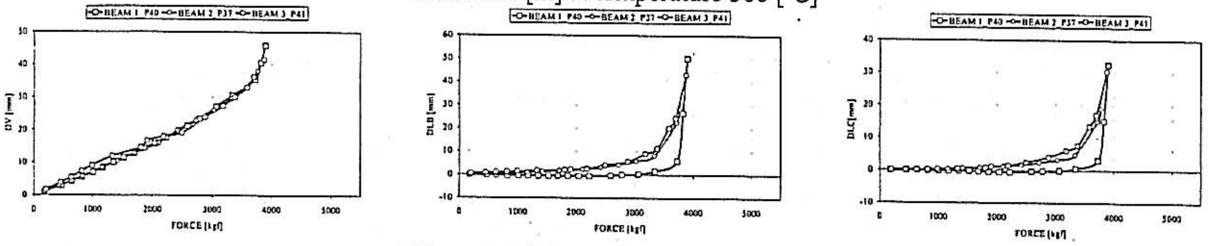
Beam 3.5 [m] at room temperature



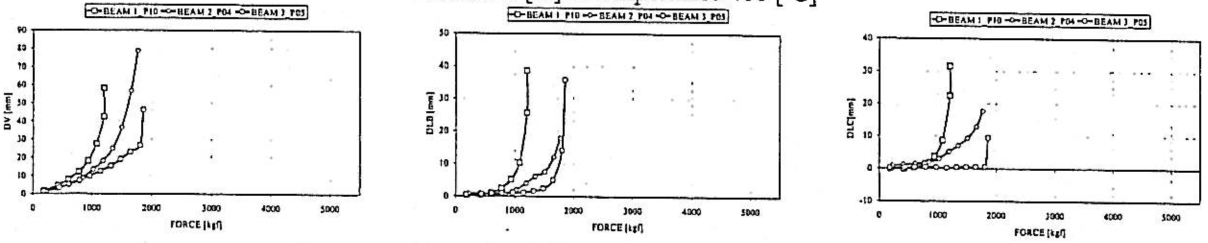
Beam 3.5 [m] at temperature 200 [°C]



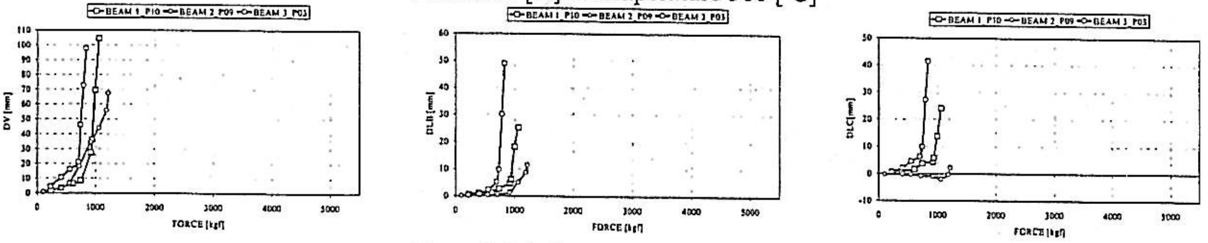
Beam 3.5 [m] at temperature 300 [°C]



Beam 3.5 [m] at temperature 400 [°C]



Beam 3.5 [m] at temperature 500 [°C]



Beam 3.5 [m] at temperature 600 [°C]

**Fig.6- Experimental results from 20[°C] to 600[°C] of a 3.5 [m] of buckling length.**

## 4 - A NEW PROPOSAL FOR A SIMPLE MODEL IN LATERAL TORSIONAL BUCKLING OF I-BEAMS.

According to the new proposal from Paulo Vila Real 1999 [1] and adopting for the lateral torsional buckling of beams the same proposal that Franssen used in 1995 [4] to represent the behavior of columns when submitted to fire conditions and with axial mechanical load, the design value to buckling resistance in fire conditions should be:

$$M_{b,fi,t,Rd} = \chi_{LT,fi} w_{pl,y} k_{y,\theta,com} f_y \frac{1}{\gamma_{M,fi}} \tag{7}$$

where $\chi_{LT,fi}$, $\phi_{LT,\theta,com}$ and $\bar{\lambda}_{LT,\theta,com}$ are given by

$$\chi_{LT,fi} = \frac{1}{\phi_{LT,\theta,com} + \sqrt{[\phi_{LT,\theta,com}]^2 - [\lambda_{LT,\theta,com}]^2}} \tag{8}$$

$$\phi_{LT,\theta,com} = \frac{1}{2}\left[1 + \alpha\bar{\lambda}_{LT,\theta,com} + (\bar{\lambda}_{LT,\theta,com})^2\right] \tag{9}$$

$$\bar{\lambda}_{LT,\theta,com} = \bar{\lambda}_{LT}\sqrt{\frac{k_{y,\theta,com}}{k_{E,\theta,com}}} \tag{10}$$

The imperfection factor $\alpha$ now is a function of a severity factor $\beta$

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

This severity factor $\beta$ should be chosen in order to ensure the appropriate safety level in the design of beams to lateral torsional buckling, and

$$\varepsilon = \sqrt{\frac{235}{f_y}} \tag{12}$$

In this formulas, $f_y$ represents the nominal yield strength of the material testing, $\bar{\lambda}_{LT}$ the relative slenderness at room temperature, $\bar{\lambda}_{LT,\theta,com}$ the relative slenderness at elevated temperature, $w_{pl,y}$ represents the plastic moment of the cross section, $k_{y,\theta,com}$ the relative coefficient of the yield strength at the temperature $\theta_{a,com}$. The partial security factor in case of fire $\gamma_{M,fi}$ should be taken as 1.0.

Comparing equations (1) and (7) we can verify that with this new proposal we do not use the empirical constant 1.2 that is used as a correction factor in the proposal of the Eurocode 3. Equations (8) and (9) are exactly the same as those defined at room temperature in [2], except that the threshold limit of 0.20 for $\bar{\lambda}_{LT}$ does not appear in equation (9). This fact changes the shape of the buckling curve, beginning at $\chi_{TL} = 1.0$ for $\bar{\lambda}_{LT} = 0.0$ but decreasing even for very low slenderness, instead of having a horizontal plateau up to $\bar{\lambda}_{LT} = 0.4$.

The lateral-torsional buckling curve varies with the yield strength due to the parameter $\varepsilon$ that appears in the imperfection factor.

### 4.1. Experimental results

A set of 120 experimental results were done in the Laboratory of Structures of the Polytechnic of Bragança, with the equipment described in the previous chapters. The geometry of the cross section was averaged from a set of specimens and it could be observed that they didn't correspond exactly to the dimensions presented by the manufacturer.

The mechanical properties were considered from 20 measures on the specimens.

The self weight from the ceramic mat, beam, and insulation material was considered to evaluate the critical buckling moment.

The results of each resistance force were recorded and graphically presented in figure 6. The last value of each experiment was considered to be the buckling resistance force.

Adopting the same value for the severity factor $\beta = 0.65$ that Paulo Vila Real used in his proposal, it can be verified that the buckling moments obtained with this simple model are in the safe side, except for the results corresponding to the smallest beams.

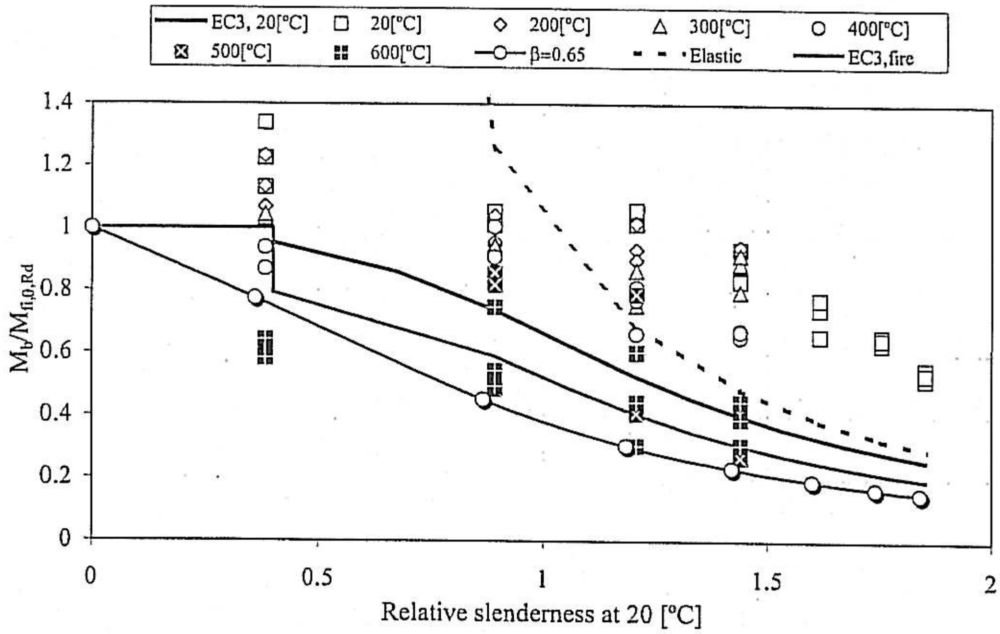


Fig. 7- Beam design curves at elevated temperatures with the new proposals.

## 5- CONCLUSIONS

The experimental results were done during several days, and the results could be influenced by a lot of parameters. First the temperature should or intended to be uniform, but in reality it does not occur, since the 4 temperature control unit only read the values in each thermocouple and action in conformity on rising and cooling of that region. Despite the beam insulation, this fact may influence the material properties and by consequence the results.

The time that each beam was exposed to the heat was almost constant but those differences may be important for the creep phenomenon.

The physical fact that Young's 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 may vanishes in the case of elevated temperatures like in the new proposal.

The beam design curve based on the reduction factor for lateral-torsional buckling in fire design situation depends, in the new proposal, on the steel grade, which is not the case in the Eurocode 3, Part 1-2.

The severity factor $\beta$ of the proposed simple calculation model has been established analyzing only the behavior of the IPE 100 profile. Further experimental results should be obtained to confirm the value of this severity factor.

## ACKNOWLEDGEMENTS

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