

CRITICAL TEMPERATURE EVALUATION FOR LATERALLY UNRESTRAINED STEEL I-BEAMS IN CASE OF FIRE

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ABSTRACT

Fire resistance may be determined by simple calculation models applied to individual members and should be based on safe results. This safety can be verified on three distinct domains, such as, temperature domain, load bearing domain and time domain. The temperature domain was chosen to show how a beam element might achieve the design limit state by lateral instability, for a specified degree of utilisation.

A set of uncouple numerical simulations using ANSYS software will be presented, regarding that mechanical deformation does not produce temperature change in the structural material (no explicit creep is considered in the mechanical model). The thermal deformation, due to temperature increase, is responsible for the introduction of mechanical stresses, which means that first is necessary to define the fire and than, for each mechanical load increment, update deformation field, under an iterative procedure (mechanical response model). The numerical model is based on geometric and material nonlinearities.

The numerical tests use shell finite elements, with initial out-of-straightness and material behaviour in accordance to the Eurocode 3. The beam element will be tested with two forks supports and with a concentrated mid span dead load Q , with increasing temperature, based on 800 [°C/h] heating rate.

A set of results for different beam lengths will be presented and demonstrate that all critical temperatures are greater than the specified value according to Eurocode 3. For critical temperature validation, a set of experimental full-scale tests will be presented.

1 INTRODUCTION

Steel structures are widely used for building construction due to the advantages of high strength, good ductility and easy fabrication and erection. Structures in fire may suffer serious damage and even collapse, reason why this accidental condition should be well studied and predicted.

At the element safety level verification, critical temperature is function of the predefined degree of utilisation. This critical temperature is determined for the collapse condition $E_{fi,d} = R_{fi,d,t}$, being $E_{fi,d}$ the design effect of actions in fire conditions and $R_{fi,d,t}$ the design resistance during fire at instant t , according to Eurocode 3 part 1.2 [1].

Actual design codes refer to fire resistance for steel construction elements as the time after which the same element ceases to fulfil the function for which it has been designed. Almost

every test is reported to a standard fire curve during numerical and experimental procedures (ISO834 [1]), and normally does not report to real fire curves, which should be the next step forward.

In this work, the critical temperature is determined for a set of unrestrained beam elements, with a mid span concentrated fixed load Q , while temperature is increased till collapse (fire), with a constant heating rate, see figure 1. Numerical simulations are based on large displacement theory, using a finite element code ANSYS to simulate this structural element in fire [2].

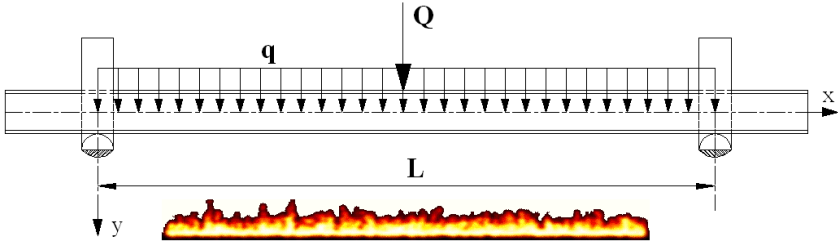


Fig. 1. Physical model under testing.

2 DESIGN LIMIT STATE FOR LATERALLY UNRESTRAINED STEEL BEAMS

Steel I beams subjected to flexural bending have greater stiffness in the web plane than in the lateral plane. Unless these members are properly braced against lateral torsional buckling, they may be subject to failure by mechanical or thermal loads, before attainment of their full in-plane capacity. Lateral torsional buckling is a structural limit state where large displacements are combined with large axial rotation, while the load (thermal and mechanical) remains almost unchanged.

The critical elastic design moment is load dependent and calculated from stationary conditions of the total potential energy, assuming the Vlassov theory for the well-known moment distribution components of concentrated load M_Q and distributed load M_q . These fundamental assumptions establish that the contour of the cross section is rigid in its own plane and there is no shear deformation in the mean surface of the section.

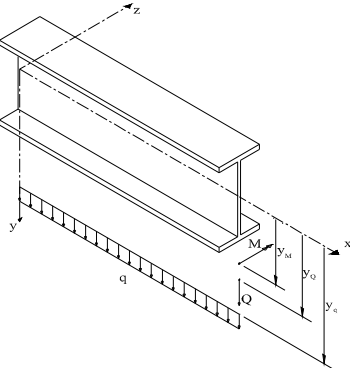


Fig. 2. Generalized beam mechanical load.

For the generalized mechanical load case presented in figure 2, where the distributed load is applied at shear centre, the elastic critical moment is obtained by the equation (1).

$$\frac{M_Q}{M_{cr,M}} + \frac{M_q}{M_{cr,M}} = 1,423 \left[\sqrt{1 + \left(0,577 \frac{P_y y_Q}{M_{cr}} \right)^2} - 1,003 \frac{P_y y_Q}{M_{cr,M}} M_q + \right. \\ \left. + 0,577 \frac{P_y y_Q}{M_{cr,M}} - 0,167 \frac{M_q}{M_{cr,M}} \right] \quad (1)$$

in which, $M_Q = QL/4$, $M_q = qL^2/8$, $P_y = \pi^2 EI_y / L^2$ and $M_{cr,M}$ being the elastic critical moment for uniform moment distribution.

Equation (1) must be used for the calculation of critical temperature, by means of an iterative procedure. This simple calculation method leads to the calculation of the degree of utilisation, equation (4), function of the beam loading case.

2.1 Lateral Torsional Buckling in Case of Fire

The Eurocode 3 [1] presents the design limit state for buckling resistance $M_{b,fi,t,Rd}$ in the load domain, for simple calculation methods, according to the equation (2).

$$M_{b,fi,t,Rd} = \chi_{LT,fi} w_{pl,y} k_{y,\theta,com} f_y \frac{1}{\gamma_{M,fi}} \quad (2)$$

$\chi_{LT,fi}$ represents the reduction factor for lateral-torsional buckling in the fire design conditions, $w_{pl,y}$ represents the plastic modulus, $k_{y,\theta,com}$ is the yield strength reduction factor, where f_y represents the nominal yield strength for the material and $\gamma_{M,fi}$ is the partial safety factor for the relevant material property.

For the temperature domain, even if the hypothesis of uniform temperature distribution is considered, Eurocode 3 [1] does not present specific direct formulae for determining the design limit temperature for this collapse mode (critical temperature). Except when considering deformation criteria or when stability phenomena have to be taken into account, the critical steel temperature $\theta_{a,cr}$ at time t for a uniform temperature distribution in a member may be determined for any degree of utilisation μ_0 at time $t=0$ using the equation (3).

$$\theta_{a,cr} = 39,19 \ln \left[\frac{1}{0,9674 \mu_0^{3,833}} - 1 \right] + 482 \quad (3)$$

The collapse mode may be described by a lateral displacement combined with an axial rotation, as represented in figure 3. The increasing load should be interpreted as mechanical or thermal load.

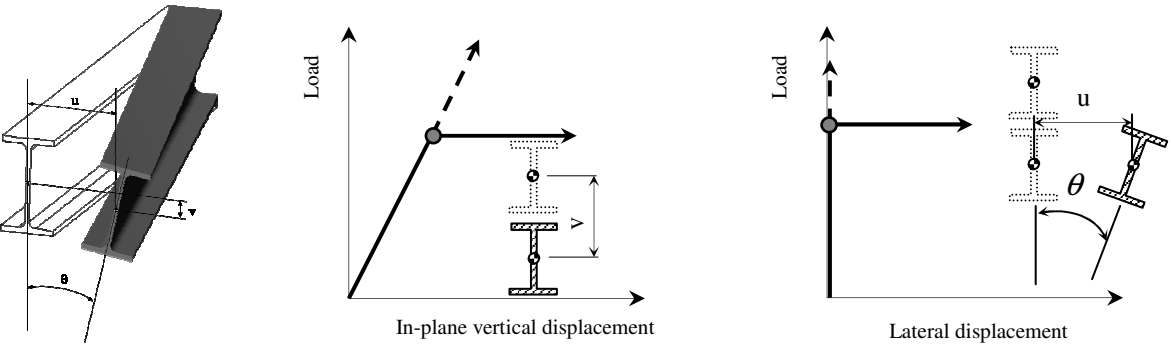


Fig. 3. Collapse mode by lateral torsional buckling.

3 NUMERICAL MODEL AND RESULTS

The numerical analysis was based on a geometric and material non-linear programme, ANSYS [2]. Steel beams have been modelled by suitable shell finite elements normally used to model, flat or warped, thin to moderately thick shell structures. This finite element (shell 181) has six degrees of freedom at each node, translations in the nodal x, y, and z directions and rotations about the same axes. The deformation shape functions are linear in both in-plane directions and presents two integration points for in-plane integration and five for normal direction. The beam cross sections were modelled with mid plane dimensions and the steel material uses an elastic- elliptic -plastic model based on Eurocode 3, see figure 4, with a yield stress of 293,2 [MPa], obtained from the tensile testes of the material characterization.

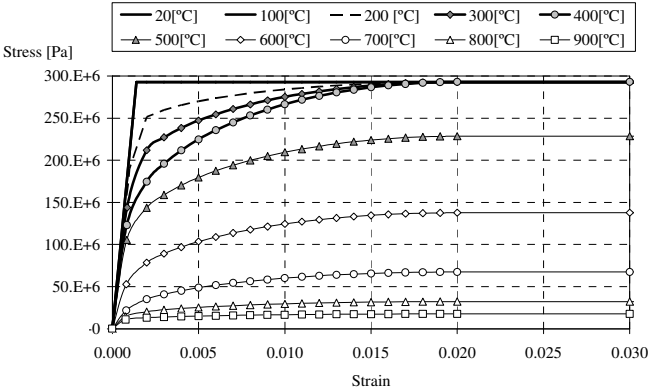


Fig. 4. Stress strain curve material at elevated temperatures.

The elastic modulus will vary with temperature in agreement with Eurocode 3, being the other mechanical properties considered constants.

Every structural real element presents initial imperfections due to fabrication processes transportation, storage and construction methods. The initial out-of-straightness imperfection causes a secondary bending moment as soon as any compression load is applied, which in

turn leads to further bending deflection and a growth in the amplitude of this bending moment lever arm. Stable deflected shape equilibrium is possible to establish until the internal compression forces does not exceed the internal moment resistance. The numerical model was implemented with an initial out-of-straightness represented by a harmonic function with maximum amplitude equal to $L/1000$.

Residual stresses will be considered, based on a theoretical distribution (bi-triangular shape), as a percentage of the material yield stress. Numerically, those initial residual stresses will be introduced on the elements integration points, as represented in figure 5.

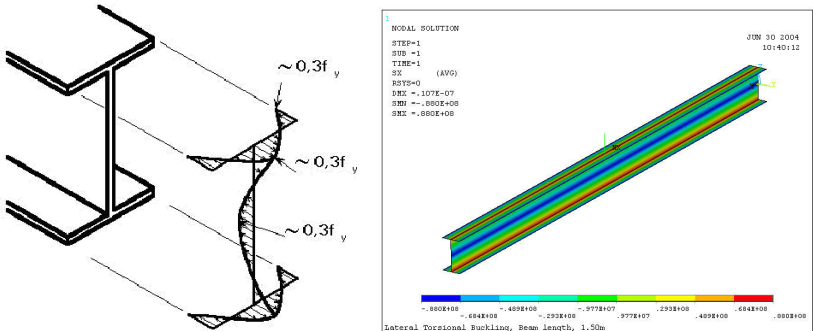


Fig. 5. Residual stress field on the element beam, applied to shell elements.

The numerical model has been implemented with four shell elements in the web and the same number over the beam flanges. The beam-ends will be modelled by two forks supports, constraining vertical and lateral displacements and letting the beam warp free.

With a material and a geometric non-linear behaviour, a thermal - incremented and interactive procedure must be implemented. Temperature will be increased with a specific heating rate of 800 [°C/h] with auto stepping, based on the elements node imposed temperature field. It will be assumed a uniform temperature over the entire beam.

The critical temperature will be determined by the last temperature increment, for which is possible to maintain the equilibrium. In figure 6 the results for critical temperature are compared with the simplified design formula. As expected, the numerical results are always greater, when compared with simplified method.

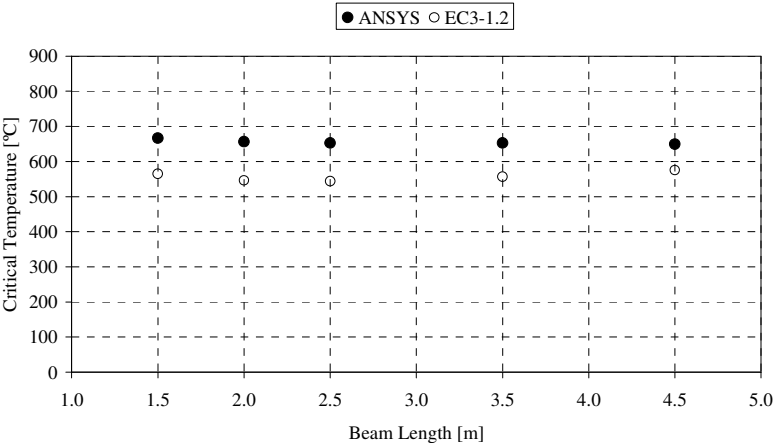


Fig. 6. Numerical and Eurocode 3 results for critical temperature of unrestrained beams.

4 EXPERIMENTAL MODEL AND RESULTS

A set of fifteen experimental full-scale tests has been carried out for beams of the European series IPE 100 with lengths varying from 1.5 to 4.5 meter. Three tests have been made for each beam length, due to statistics requirements. The beams were electrically heated by means of ceramic mat elements, heated by a power unit of 70 kVA. A ceramic fibre mat was used around the beam and heating elements in order to increase the thermal efficiency.

A simply supported beam with fork supports shown in figure 7 has been studied. In this figure, q represents the self-weight of the beam and the additional distributed load due to the weight of the insulation mat and electro-ceramic resistances.

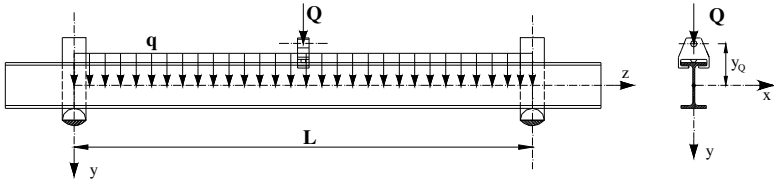


Fig. 7. Experimental model under testing.

4.1 Experimental Set-up

The experimental set-up is represented in figure 8. The reaction frame (1), with two fork supports (4), will help to test the beams (9), using a dead load system (2). The load will be applied on the top flange, at a distance $y_Q = 0,105 [m]$, by means of a balance system represented by (3) that will maintain the load in vertical position.

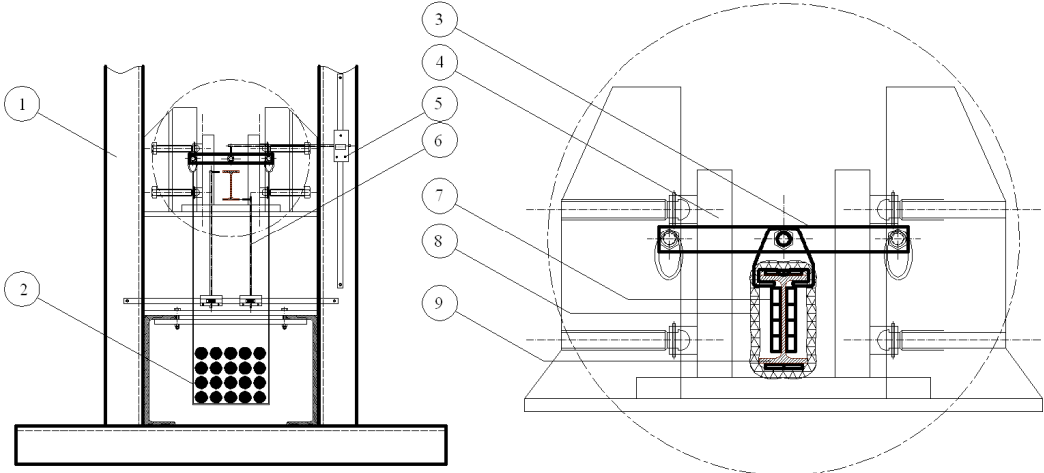


Fig. 8. Experimental set-up.

The beam will be heated by means of an electro ceramic resistance (7), protected by an insulation mat (8). Displacements will be followed by three digital measuring rules (5,6) for lateral and vertical mid span measurements.

4.2 Experimental Procedure

Every tested beam was dimensionally controlled with a laser beam method to measure the initial out-of-straightness. This measured imperfection was considered as a global imperfection and in agreement with the specified value for numerical simulations.

A set of 11 tensile strength specimens was extracted from the web of the beams and tested according to national standards NP EN 10002-1 [3]. The yield strength $f_y = R_{eH} = 293 [MPa]$ and the elastic modulus $E = 210 [GPa]$ have been determined and compared with the inspection certificate of the profile producer.

The beams were loaded for a specific degree of utilisation in fire conditions μ_0 . The mechanical load was specified for typical practical values (~60%). The degree of utilisation is defined as the ratio of the design load effect relative to the beam resistance in fire conditions for time equal to zero, as represented in equation (4).

$$\mu_0 = \frac{E_{fi,d}}{R_{fi,d,0}} = \frac{M_Q + M_q}{M_{b,fi,0,Rd}} \quad (4)$$

This coefficient is function of the load type, which on this case provides a triangular flexural moment distribution. The resistance should be calculated for the expected collapse mode (lateral torsional buckling). Table 1 presents the mechanical load applied in each beam and the resultant critical temperature obtained by an iterative procedure according to the Eurocode 3 formulation.

| Beam effective length [m] | $q [N/m]$ | $Q [N]$ | $E_{fi,d} = \frac{QL}{4} + \frac{qL^2}{8}$ | $\mu_0 = \frac{E_{fi,d}}{R_{fi,d,0}}$ | EC3-1.2 $\theta_{a,cr} [^\circ C]$ |
|---------------------------|-----------|---------|--|---------------------------------------|------------------------------------|
| 1,5 | 134,38 | 6086,12 | 2320,09 | 56 % | 565,15 |
| 2,0 | 123,00 | 4315,52 | 2219,26 | 63 % | 546,31 |
| 2,5 | 116,18 | 3043,06 | 1992,68 | 64 % | 543,64 |
| 3,5 | 118,14 | 1521,53 | 1512,24 | 59 % | 556,85 |
| 4,5 | 111,64 | 772,54 | 1151,69 | 53 % | 575,48 |

Table 1. Mechanical load and calculated degree of utilisation.

After mechanical load, the temperature load is increased by means of electro ceramic resistance, with the specified heating rate of 800 [°C/h], till collapse.

4.3 Experimental Results

The ultimate limit states should correspond to the maximum load bearing capacity but, typically, in fire design conditions may corresponds to the “near-maximum” load bearing capacity intend to avoid possible damage on furnaces, caused by test specimens collapse, as defined in British standard BS476. For load bearing horizontal elements, the test specimen shall be deemed to have failed if is no longer able to support the test load. For the purpose of this standard the ultimate design limit is first exceeded if the deflection limit $L/20$ is achieved or if the rate of deflection reaches $L^2/(9000 d)$, being d the distance from the top of the structural section to the bottom of the design tension zone. However, this rate of deflection limit shall not apply before a deflection limit of $L/30$ is exceeded [4], [5]. Due to constrain dimensions of experimental set-up and reaction frame, those limits could not be obtained, nevertheless the collapsing mode has been achieved by lateral torsional buckling, see figure 9.



Fig. 9. Experimental tests. Three different phases.

For each beam length, vertical and lateral displacement (top and bottom flange) has been measured, as can be seen in the next figure, for the smaller and larger beam slenderness.

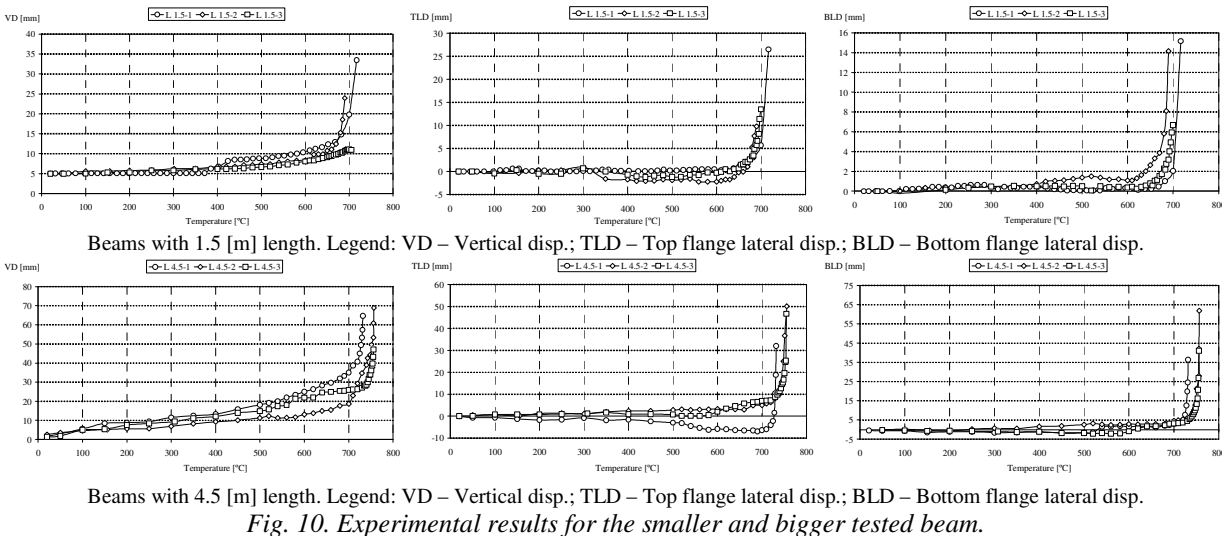


Fig. 10. Experimental results for the smaller and bigger tested beam.

Critical temperature has been considered for the last measuring point, for which a small temperature increment produces a large lateral displacement. In figure 11, all measured

critical temperatures have been registered, being the experimental results slightly greater than the simple calculation formula [1]. This fact can be related with insufficient insulation near supports, introducing an additional stiffness and due to the non-uniform temperature distribution along the beam length.

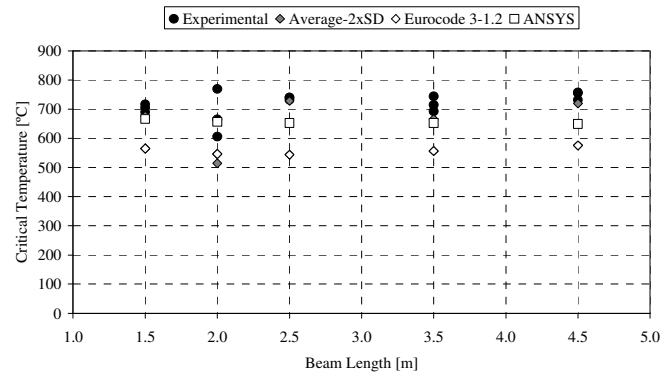


Fig. 11. Critical temperature results for all tested beams.

5 CONCLUSIONS

Critical temperature has been determined for several laterally unrestrained beams, based on numerical and experimental tests. The tested beams were subjected to a constant mechanical load and then subjected to an increasing temperature, approaching to the accidental fire conditions.

For each set of experimental tests the results shows small scattered data points. Both numerical and experimental results lead to higher critical temperatures in comparison with the simplified design calculation procedure for this instability phenomenon. Possible reasons for the experimental results are related to the non-uniform temperature distribution along the beam, increasing stiffness effect and also due to the shape of fork supports, introducing a restriction to lateral rotation with a friction effect.

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