NUMERICAL MODELLING OF LATERAL TORSIONAL BUCKLING FOR PARTIALLY ENCASED STEEL BEAMS AT ELEVATED TEMPERATURES

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ABSTRACT

Partially encased beams are composed structural elements widely used for industrial and commercial buildings with increasing significance. Composite beams may be designed for service load conditions without the collaborative contribution of slabs. Instability problems may occur because concrete may not have the age to resist and also because the concrete may slip over steel, crack or crush. Lateral torsional buckling is an instability phenomenon that may occur in these situations.

This paper presents a material and geometric non linear finite element model for determining lateral torsional buckling resistance of partially encased beam without encasement reinforcement in fire conditions. The steel part of the composed section will be modelled by finite shell element, concrete part by three dimensional finite solid elements and the bond contact with finite non linear spring elements, from Ansys. Elevated temperatures will be applied in both four sides of the cross section along the beam, based on the ISO834 standard fire curve.

Failure of concrete will also be predicted when subjecting partially encased beams to fire conditions.

1. INTRODUCTION

Steel beam with partial encasement represents a composite section with two different materials, with distinct mechanical behaviour, being usually used in construction for increasing fire resistance, see figure 1.

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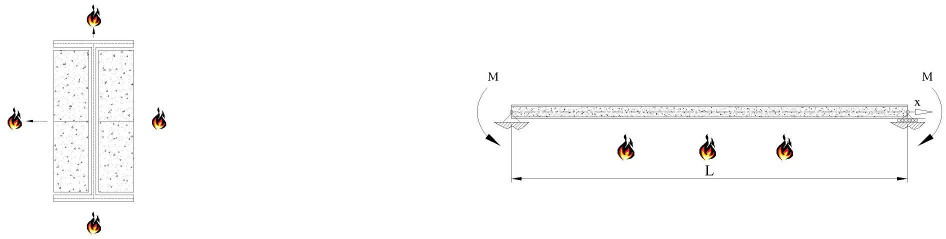


a) Composed cross section model. b) Beam load model.

Fig. 1 – Partially encased steel beam with concrete.

Concrete slabs may not transmit the stabilizing effect to prevent lateral torsional buckling (LTB) of partially encased beams. Also, concreting the steel beam within 7 or 8 days provides less torsional stiffness than after 28 days [1]. This instability phenomenon is responsible for a simultaneous lateral displacement and cross-section rotation. This means that bending around the minor axis and torsion about the longitudinal axis of the element are involved.

Experimental tests conducted at room temperature were presented by the Technical University of Berlin [2] and other experimental and numerical results were presented during the European research project for technical steel research, at CTICM and LABEIN [3]. This work intends to be the preliminary phase of a full scale test program in fire conditions, complemented with numerical research and intends to deal with the interaction between concrete and steel, finding failure of concrete that may occur during instability, in these conditions.

Non linear numerical results of partially encased concrete beams at room temperature and in fire conditions will be presented for the case of uniform bending load, see figure 1.

Incremental mechanical load will be applied at room temperature conditions to determine design buckling resistance.

A transient non-linear thermal analysis will be subsequently followed by a non linear static analysis to address the ultimate limit state. Beams will be subjected to constant mechanical load (50% of the plastic resistant moment) at fire conditions.

2. LATERAL TORSIONAL BUCKLING (LTB)

Accidental fire conditions reduce the load bearing capacity and increase the risk of failure by lateral instability. The partial concrete protection may not be sufficient to provide stabilizing effect during the increase of steel temperature. Ultimate LTB resistance must be checked in the designing phase in order to get the assurance that structures are appropriately safe and resistant with regard to accidental loading conditions.

2.1 Critical moment

The ideal behaviour of a perfectly straight simply supported beam with perfectly elastic material behaviour, subjected to bending about major axis and prone to LTB is characterized with elastic critical moment (M_{cr}). This value corresponds to the load level at equilibrium bifurcation [3].

For LTB, the critical moment may be determined assuming the validity of the elastic theory (energy method) and some assumptions regarding the concrete strength contribution. Torsion constant and the moment of inertia may be calculated for the composed section based on the characteristic values of steel part and based on the reduced characteristic values of concrete part, as represented in the next expressions [2].

$$\begin{aligned} J &= J_a + J_{c,red} \\ I_z &= I_{za} + I_{zc,red} \end{aligned} \tag{1}$$

where J and I_z represent the torsion constant and the moment of inertia of the composed section. $J_{c,red}$ and $I_{zc,red}$ represent the reduced torsion constant and the reduced moment of inertia of concrete, while J_a and I_{za} represent the torsion constant and the moment of inertia of steel. The concrete part is accounted for by reducing this part to an equivalent steel part, $J_{c,red}$ and $I_{zc,red}$, using equation 2.

$$\begin{aligned} I_{zc,red} &= I_{zc} E_c / E_a \\ J_{c,red} &= J_c \, G_c / G_a \end{aligned} \tag{2}$$

Taking into consideration only the compressive part of concrete, see figure 2, both geometric properties may be determined according to the plastic compressive zone, defined by e_{pl}.

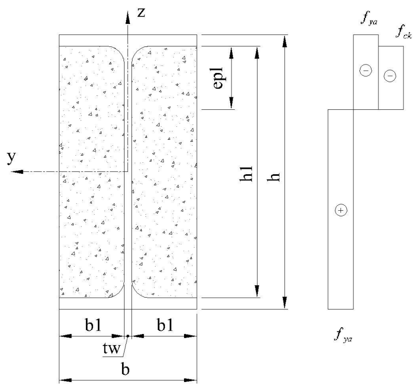


Fig. 2 – Notation for material plastic section.

$$e_{pl} = \frac{2 f_{ya} \, t_w \, h_1 / 2}{2 f_{ya} \, t_w + 2 b_1 \beta_R} \tag{3}$$

where f_{ya} and β_R represent the design strength of steel and concrete.

The geometric properties of concrete may de determined according to the previous expression.

$$\begin{aligned} \alpha &= \left[1 - 0.63 \, e_{pl} / (b - t_w)\right] / 3 \\ J_c &= (b - t_w) e_{pl}^{\,3} \alpha \\ I_{z,c} &= e_{pl} (b - t_w)^3 / 12 \end{aligned} \tag{4}$$

Assuming constant stiffness values along the beam length, considering the influence of the warping restrain small, the influence of moments of inertia much smaller than the influence of torsion constant, assumptions may be used to calculate the critical moment. The warping constant of the concrete part is neglected and a symmetric section may be considered [2].

Flexural and torsional stiffness are determined in accordance to the previous equations. Critical moment may be determined taking into account the loading conditions, the moment distribution and the lateral restraints. For uniform bending moment and simple supported beams the critical is defined by equation (5).

$$M_{cr} = \frac{\pi}{L}\sqrt{(EI_z)\left(GJ + \frac{\pi^2 EI_w}{L^2}\right)} \tag{5}$$

2.2 Plastic moment

The full plastic moment is well known for the case of steel beams. For partially encased beams only the plastic compression zone of concrete will be considered, in accordance to the represented model to determine plastic moment without rebar reinforcement [2].

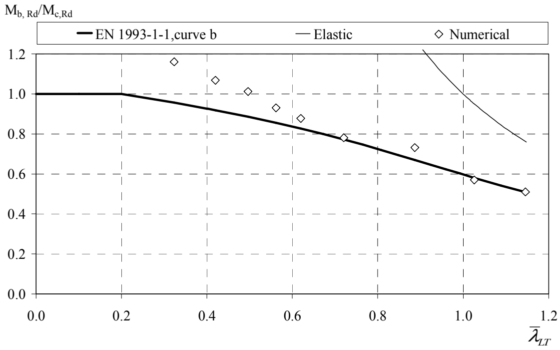
$$M_{pl} = M_{pl,a} - \frac{2 f_{ya} t_w (0.5h_1 - e_{pl})^2}{2} + \beta_R 2 b_1 e_{pl}(0.5e_{pl} + 0.5h_1 - e_{pl}) \tag{6}$$

$M_{pl,a}$ represents the full plastic moment for the steel section.

2.3 Design buckling resistance at room temperature

The real behaviour of partially encased steel beams is significantly different from the ideal. Buckling develops since the very beginning of the loading by equilibrium divergence. The ultimate LTB moment resistance ($M_{b,fi,Rd}$) corresponds to the maximum load bearing capacity. The non-dimensional beam slenderness, $\overline{\lambda}_{LT}$, is the governing parameter for LTB. The larger slenderness of a beam corresponds to a smaller ultimate LTB resistance. The reduction factor χ_{LT} should be derived from a specific buckling curve. No allowance for LTB will be considered for non-dimensional slenderness smaller than $\overline{\lambda}_{LT,0} = 0.2$.

Figure 3 represents the design buckling resistance for partially encased steel S235 IPE200 beams with C20/25 concrete, using the simplified method presented in Eurocode 3. Numerical results, obtained at room temperature, are also presented for specific beam lengths.



$\overline{\lambda}_{LT}$	$M_{b,Rd} / M_{c,Rd}$
0,32	1,16
0,42	1,07
0,49	1,01
0,56	0,93
0,62	0,88
0,72	0,78
0,89	0,73
1,03	0,57
1,15	0,51

a) Design buckling resistance curve. b) Numerical results.

Fig. 3 – Lateral torsional buckling design for partially encased steel with concrete.

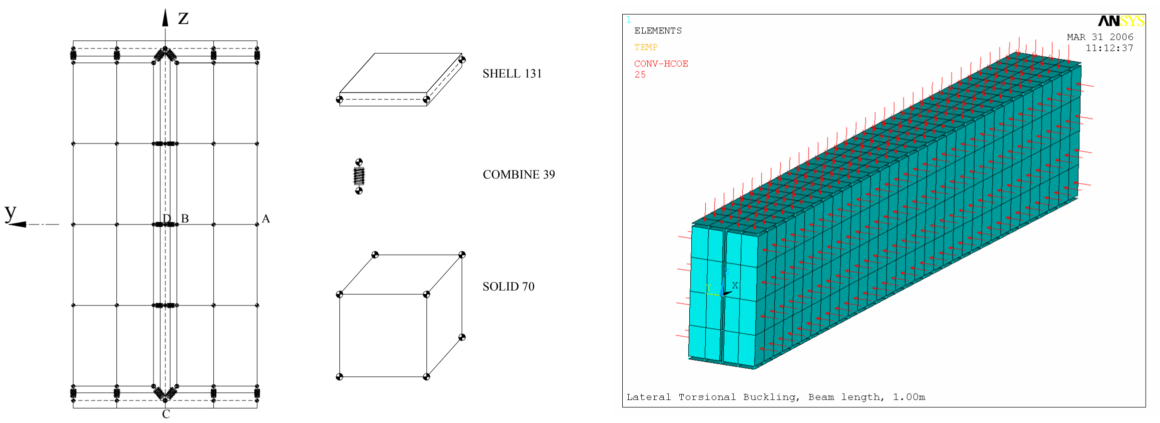
3. THERMAL NUMERICAL MODEL

Partially encased beam will be subjected to fire conditions, according to the standard fire nominal curve, ISO 834. Convective heat flux will be applied to the composite member, depending on the bulk temperature and constant convective coefficient $\alpha_c = 25\left[W/m^2K\right]$. Radiative heat flux will depend on the assumed configuration factor, resultant emissivity, $\varepsilon_{res} = 0.7$, considered for both exposed steel and concrete surfaces, and on the radiation temperature, considered equal to the bulk nominal temperature [4].

The procedure for determining the temperature development in this type of composite members may be based on advanced calculation models. Non linear transient analysis will be used, based on a full incremental time procedure. All the heat flux exchanges, between concrete and steel, consider both materials in perfect contact.

3.1 Numerical model

Three dimensional model based on ANSYS shell131, solid70 and combine39 finite elements were used to simulate thermal behaviour of partially encased steel beams with concrete in fire conditions, see figure 4.



a) Finite elements used to model cross section. b) Finite element representation.

Fig. 4 – Cross section and mesh representation for thermal model.

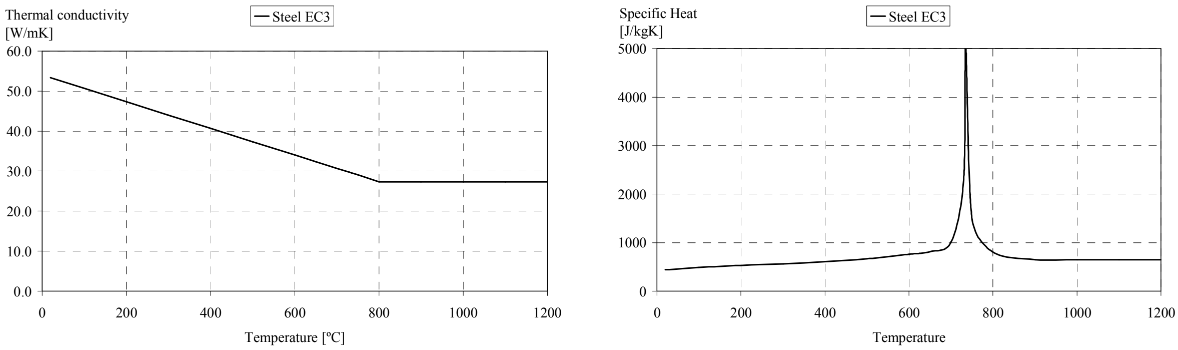
Finite shell element (shell131) is a three-dimensional layered shell with four nodes having in-plane and thru-thickness thermal conduction capability. The conducting shell element is applicable to three dimensional transient thermal analyses. Shell131 generates temperatures that can be passed to structural shell elements in order to model the thermo – mechanical behaviour. The shape functions are linear for in-plane interpolation with 2x2 integration scheme, while for thickness (assuming no temperature variation) the shape functions are assumed constant with 1 integration point.

Finite solid element (solid70) has a three-dimensional thermal conduction capability. This element has eight nodes with a single degree of freedom (temperature at each node). The element is used to model concrete three dimensional transient thermal analyses and must be replaced by an equivalent structural element to model thermo-mechanical behaviour. The shape functions are linear in each orthogonal direction and possess a 2x2x2 integration scheme.

Finite combine element (combine39) is a unidirectional finite element defined by two nodes with nonlinear generalized temperature variation – heat flux capability. The element behaviour is defined by the generic curve temperature versus heat flux. The element has no mass or thermal capacitance. The full contact area of steel and concrete was modelled by this non linear spring element, connecting node to node, between steel finite shell element and concrete finite solid element. The bond thermal model considers perfect contact, allowing for all thermal flux to pass throughout the interface in both directions. The heat flux – temperature variation curve is defined constant and does not input any thermal resistance across interface.

3.2 Material behaviour

Non-linear unsteady state analysis requires the knowledge of conductivity and specific heat variation of steel with temperature. According to EC3 - part 1.2 [5], for carbon steel this properties varies with temperature, as represented in figure 5. Specific mass is assumed to be constant and equal to 8750 [kg/m^3].



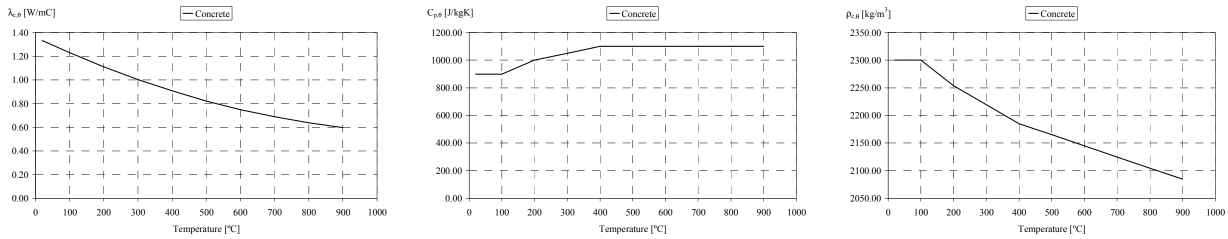
a) Thermal conductivity b) Specific heat.

Fig. 5 – Thermal behaviour of carbon steel S235.

Thermal conductivity of concrete, λ_c, may be obtained from Eurocode 2 [6], between lower and upper limit values. The lower limit of thermal conductivity for normal weight concrete was adopted.

Specific heat, $C_p(\theta)$ of dry concrete (u = 0%), depends on temperature and was obtained from the siliceous aggregates [6], according to figure 6.

The variation of the specific mass of concrete with temperature is influenced by water loss during the heating process and is defined as follows [6].

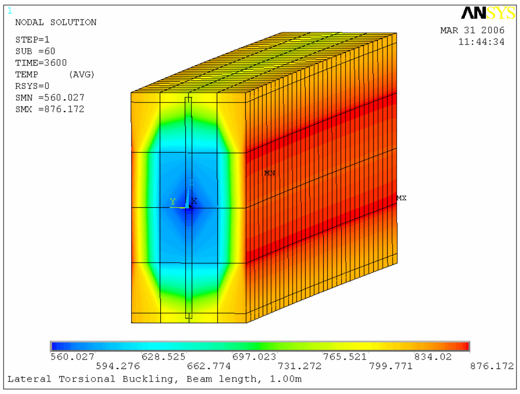


a) Thermal conductivity. b) Specific heat c) Specific mass

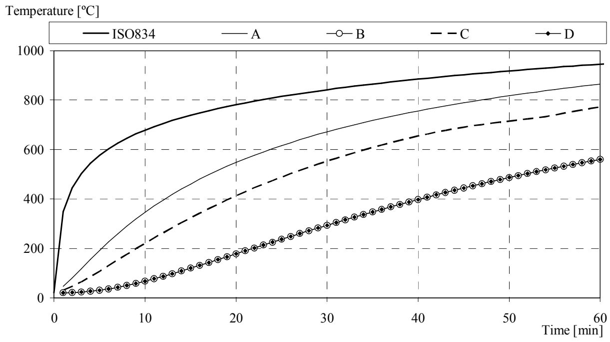
Fig. 6 - Thermal properties of concrete C20-25.

3.3 Results

Results may be presented in time domain. Figure 7 represents the temperature variation across the partially encased section. The steel web receives heat, principally, from both steel beam flanges and less from the adjacent encased concrete.



a) Temperature profile section for time equal to 60 minutes.



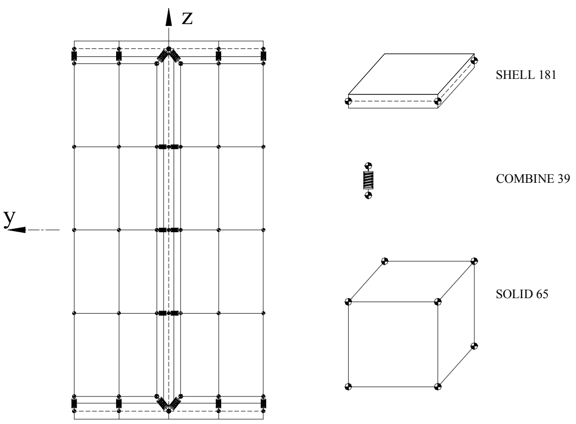
b) Temperature evolution for points A, B, C and D (referred to figure 4).

Fig. 7 – Finite element thermal transient results.

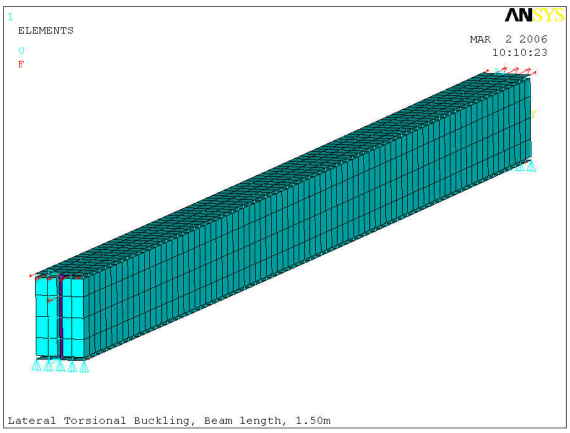
4. THERMO-MECHANICAL NUMERICAL MODEL

4.1 Numerical model

Three dimensional model based on ANSYS shell181, solid65 and combine39 finite elements were used to simulate thermo-mechanical behaviour of partially encased steel beams with concrete in fire conditions, see figure 8.



a) Finite elements used to model cross section.



b) Finite element shape representation.

Fig. 8 – Cross section and mesh representation for thermo-mechanical model.

Finite shell element is suitable for analyzing thin to moderately-thick shell structures. It is a 4-node element with six degrees of freedom at each node: translations in the x, y, and z directions, and rotations about the same axis. This element is also well-suited for material and geometric non-nonlinear applications. Bilinear elements, when fully integrated (2x2, in plane), are too stiff for in-plane bending, nevertheless, full integration scheme was adopted because this element uses the method of incompatible modes to enhance the accuracy in bending-dominated problems [7]. This element is associated with linear elastic and elasto-plastic material properties, being the von Mises isotropic hardening plasticity model used with multilinear isotropic hardening.

Finite solid element is used to model concrete and is capable of cracking in tension and crushing in compression. The element is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. This element presents linear shape functions with an integration scheme of 2x2x2.

The non linear spring is a uniaxial element defined by two nodes with nonlinear generalized force-deflection capability with large displacement. Figure 9 represents the behaviour of the bond model between concrete and steel, representing the force versus relative displacement. The longitudinal option is an uniaxial tension-compression element with up to three degrees of freedom at each node: translations in the nodal x, y, and z directions. No bending or torsion is considered. The full contact area of steel and concrete was modelled by this non linear spring element, connecting node to node finite shell element and finite solid element. The bond mechanical model was considered equal to the behaviour of the push out tests obtained for concrete –filled steel tubes [8]. When the last data point force is exceeded, the previous slope is maintained.

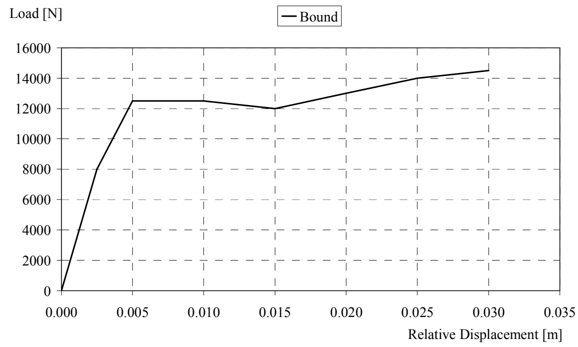


Fig. 9 – Bond interface between concrete and steel.

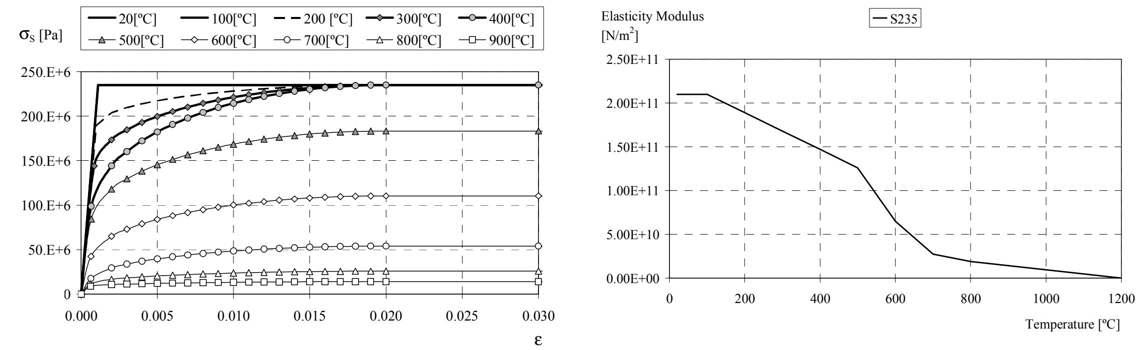
The assumption of this bond model ensures that once the peak level force is reached, concrete will travel along the steel with practically no increase in force (curve almost parallel to horizontal axis). This approach may be questionable due to the fact that bond behaviour may be considered different in tension compared to the compression zones of the flexural member. The element behaviour follows the represented curve in tension and in compression.

Structural elements always present 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. Stable deflected shape equilibrium can be established until the internal compression forces do not exceed the internal moment resistance. The numerical model was implemented with an initial out-of-straightness represented by a harmonic function with $L/400$ of maximum amplitude, to account for global imperfections.

A constant mechanical bending moment is applied, corresponding to 50% of the section plastic moment, and thermal load resulting from fire conditions applied for each time step. A non linear static analysis was undertaken to address the ultimate limit state.

4.2 Material behaviour

Steel stress-strain relation is based on an elastic- elliptic -plastic model [5], see figure 10. The yield stress at room temperature was considered equal to the characteristic value of steel S235. The elasticity modulus varies with temperature as represented.

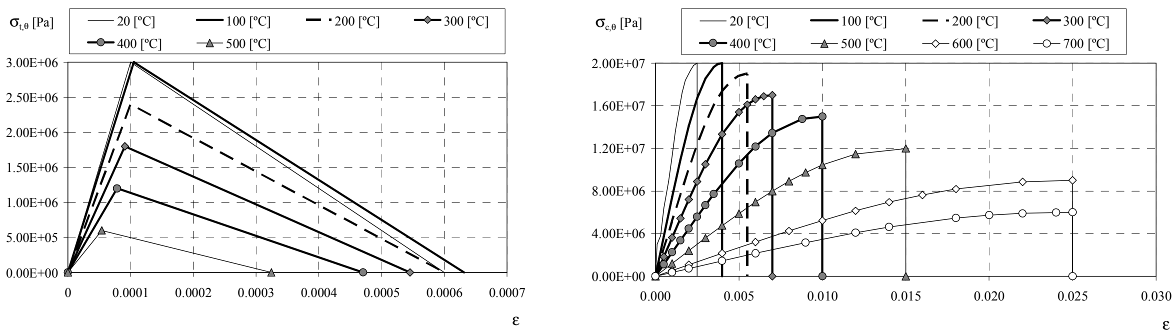


a) Stress – strain versus temperature. b) Elastic modulus versus temperature.

Figure 10 – Mechanical behaviour of steel S235.

The concrete material is capable of directional integration point cracking and crushing besides incorporating plastic behaviour. Cracking is allowed in three orthogonal directions at each integration point. If cracking occurs at an integration point, the cracking is modelled through an adjustment of material properties which effectively treats the cracking as a "smeared band" of cracks, rather than discrete cracks. In addition to cracking and crushing, the concrete may also undergo plasticity, with the Drucker-Prager failure surface being most commonly used. In this case, the plasticity is verified before the cracking and crushing checks occurs. This material model predicts either elastic behaviour, cracking behaviour or crushing behaviour. If elastic behaviour is predicted, the concrete is treated as a linear elastic material. If cracking or crushing behaviour is predicted, the stress-strain elastic matrix is adjusted for each failure mode. The concrete material is assumed to be initially isotropic.

The presence of a crack failure mode at an integration point is represented through modification of the stress-strain relations by introducing a plane of weakness in a direction normal to the crack face. If stress relaxing is considered in tension, the secant modulus works with adaptive descent and diminishes to 0.0 as the solution converges, see figure 11. A multiplier coefficient of 1.0 was used for determining the amount of tensile stress relaxation.



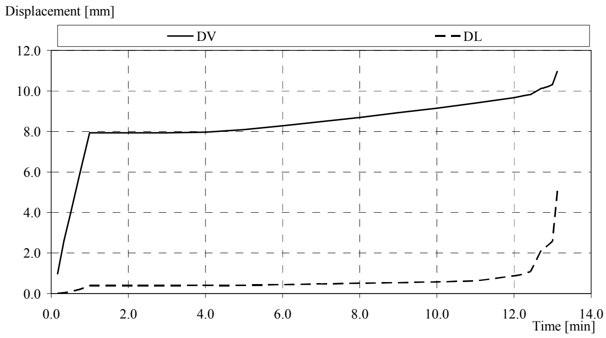
a) Stress-strain behaviour of concrete in tension. b) Stress-strain behaviour of concrete in compression

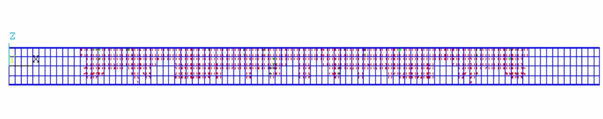
Fig. 11 - Strength for concrete material.

The presence of a crushing failure mode at an integration point is defined as the complete deterioration of the material structural integrity. Under these conditions, material strength is assumed to have degraded to an extent such that the contribution to the stiffness of an element at the integration point can be ignored.

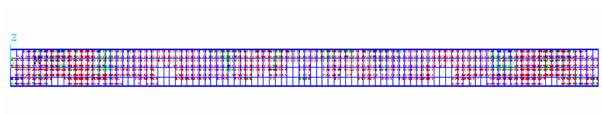
4.3 Results

The fire resistance time will be determined for the last time increment, which a small temperature increment produces a large displacement and is possible to sustain the equilibrium. As represented in figure 12, concrete cracks after the mechanical load is applied and a progressive failure is verified as the temperature rise.





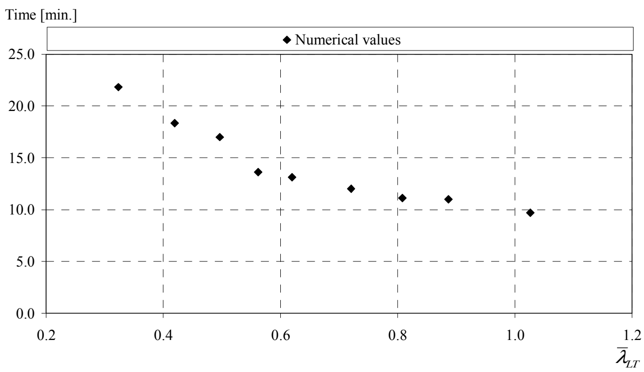
b) Initial cracks after the applied mechanical load.



a) Lateral and vertical displacements. c) Cracking and crushing of concrete at ULS.

Fig. 12 – Results for a beam length 3 [m].

In figure 13, the fire resistance time is presented for different beam slenderness values. Fire resistance decreases as the beam slenderness increase, for the same initial mechanical load.



L [m]	$\bar{\lambda}_{LT}$	t [min]
1.0	0.32	21.83
1.5	0.42	18.35
2.0	0.50	17.00
2.5	0.56	13.61
3.0	0.62	13.12
4.0	0.72	12.00
5.0	0.81	11.12
6.0	0.89	11.00
8.0	1.03	9.70

Fig. 13 – Numerical values of fire resistance time.

5. CONCLUSIONS

A non linear finite element model for determining lateral torsional buckling resistance of partially encased beams in fire conditions was presented. A special attention was considered to the bond model between concrete and steel. The model predicts the concrete failure by cracking and crushing. Fire resistance of partially encased beams subjected to a constant uniform bending moment and to an increasing temperature has been determined.

The results have shown that, with the same mechanical load, as the beams slenderness increases the fire resistance time decreases.

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