ULTIMATE LIMIT STATE DESIGN FOR LATERAL TORSIONAL BUCKLING OF PARTIALLY ENCASED STEEL BEAMS

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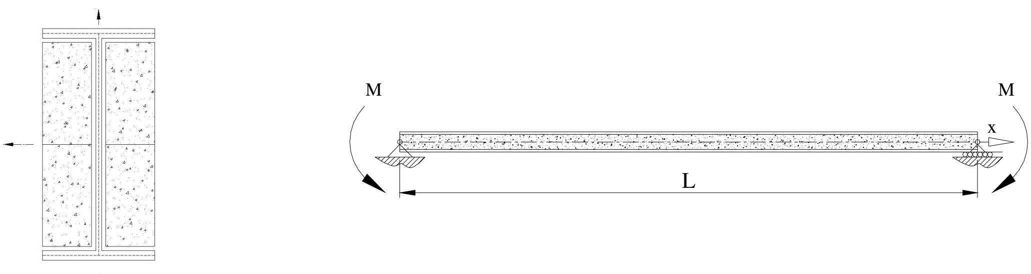
SYNOPSIS

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 concrete may slip over steel, crack or crush. Lateral torsional buckling is an instability limit state that may occur in these situations.

This paper presents Ansys material and geometric non-linear finite element model for the design of lateral torsional buckling resistance of partially encased steel beam without encasement reinforcements at room temperature. The steel part of the composed section will be modelled by shell, concrete by three dimensional solids and the bond contact with non-linear spring finite elements. Failure of concrete will also be predicted when the beam is subjected to a constant bending moment.

INTRODUCTION

Steel beam with partial concrete encasement represents a composite section with two or more different materials and types of construction with distinct mechanical properties and mechanical behaviour, see figure 1. Normally partially encased beams are used with reinforcement rebars and with or without structural link to slabs.



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

Fig. 1 – Partially encased steel beam with concrete.

Slabs may not transmit the stabilizing effect to prevent lateral torsional buckling (LTB). Also, concreting the steel beam within 7 or 8 days provides less torsional stiffness than after 28 days (Boissonade et al, 2004). 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 (Lindner et al, 2002) and other experimental and numerical results were presented during the European research project for technical steel research, at CTICM and LABEIN (EU, 2002), both representing the state of the art for this subject.

This work intends to be the preliminary phase of a full-scale experimental 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. Numerical results of unrestrained partially encased steel beams will be presented for the uniform bending loading case, see figure 1. Non-linear geometric and material static analysis will address this ultimate limit state, using an incremental load procedure to determine buckling resistance.

LATERAL TORSIONAL BUCKLING (LTB) FOR PARTIALLY ENCASED BEAMS

Partial concrete encasement may not be sufficient to provide stabilizing effect during the service conditions. Ultimate LTB resistance must be checked during the design phase in order to get the assurance that structures are appropriately safe. To achieve this goal, advanced calculation methods may be used for the assessment of safety in structures and in particular of structural elements. In order to ensure that ultimate limit requirements are fulfilled, it is necessary to predict failure of each type of material and elements during the design process of a building.

Lateral torsional buckling mode of instability should be checked according to current European design standards (CEN, 2004-2005), being the critical moment and the plastic moments two important design values to be calculated. This section provides simple design formulae for both.

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. 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 (Lindner et al, 2002).

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

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.

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

The geometric properties of concrete may de determined according to the previous expression, considering the following relations.

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

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

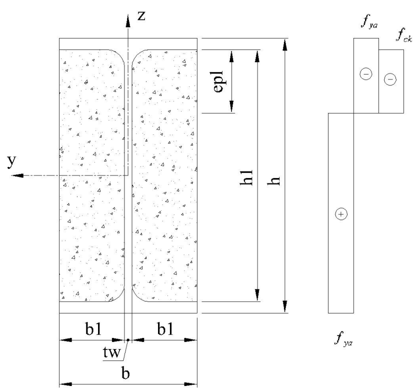


Fig. 2 – Notation for material plastic section.

$$e_{pl} = \frac{2 f_{ya}\, t_w\, h_1 / 2}{2 f_{ya}\, t_w + 2 b_1 \beta_R} \quad (4)$$

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 (Lindner et al, 2002). Flexural and torsional stiffness is determined in accordance to the previous equations. Critical moment may be determined taking into account the type of loading conditions, the moment distribution and lateral restraints. For uniform bending moment and simple supported beams the critical moment is defined by equation (5).

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

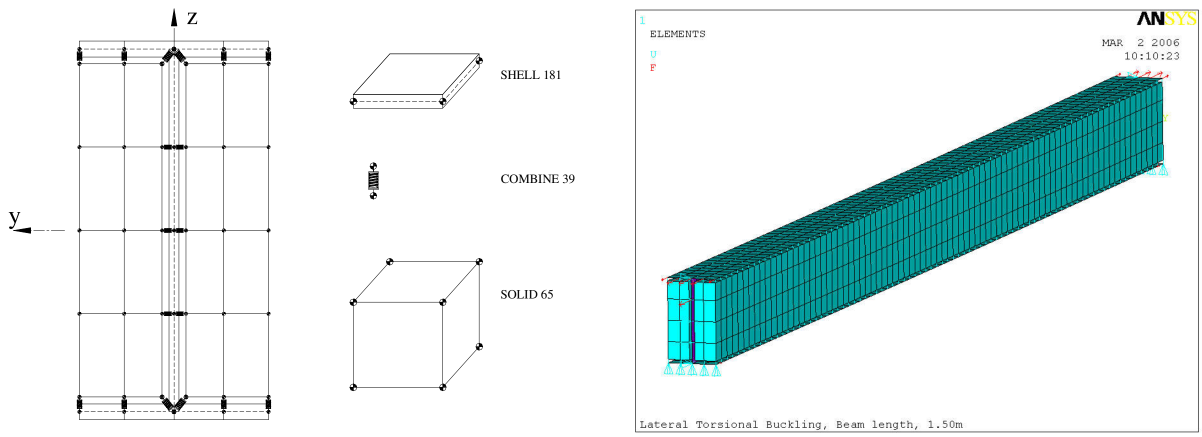
Full plastic moment, M_{pl} is well known for the case of steel beams. For partially encased beams only the plastic compression zone of concrete will be considered, in accordance to the represented model to determine plastic moment without rebar reinforcement (Lindner et al, 2002).

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

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

NUMERICAL MODEL FOR PARTIALLY ENCASED BEAMS

Three-dimensional model based on ANSYS shell181, solid65 and combine39 finite elements were used to simulate the numerical behaviour of partially encased steel beams with concrete at room temperature conditions, see figure 3.



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

Fig. 3 – 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 (Simo et al, 2002). This element is associated with linear elastic and elasto-plastic material properties, being the von Mises isotropic hardening plasticity model used with multi-linear isotropic hardening.

Finite solid element is used to model concrete and is capable of cracking in tension and crushing in compression. Solid 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 4 represents the behaviour of the bond slip model between concrete and steel, representing the force versus relative displacement. The longitudinal option is a 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.

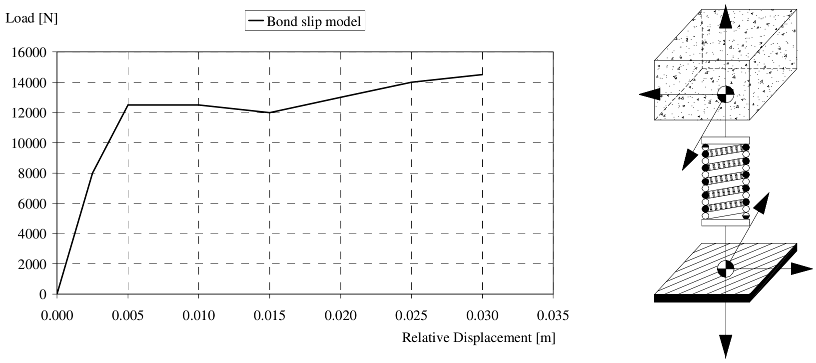


Fig. 4 – Bond interface between concrete and steel.

The full contact area between steel and concrete was modelled by this non-linear spring element, connecting node-to-node finite shell element and finite solid element. The bond slip model was considered equal to the behaviour of the push-out tests obtained for concrete filled steel tubes, (Brahmachari, 1997). When the last data point force is exceeded, the previous slope is maintained. The assumption of this bond slip model ensures that once the peak level force is reached, concrete will travel along the steel with practically no increase in force. This approach may be questionable due to the fact that bond behaviour may be considered different in tension zones 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.

An incremental iterative procedure is based on a non-linear static analysis, up to the last increment in force in which is possible to sustain the equilibrium, using a convergence numerical procedure, based on displacements and rotations.

The material behaviour corresponds to both simplified models proposed by Eurocodes, assuming that steel stress-strain relation is based on an elastic-perfect plastic model, (CEN,

2005) see figure 5. The yield stress at room temperature was considered equal to the characteristic value of steel S235.

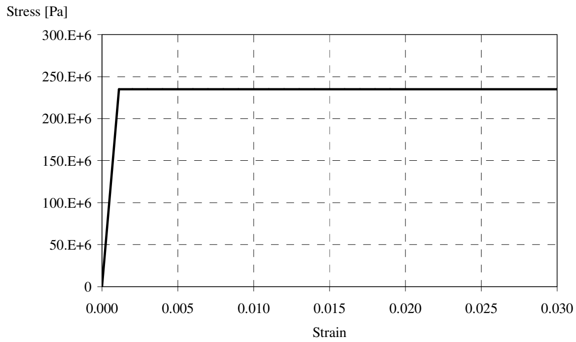
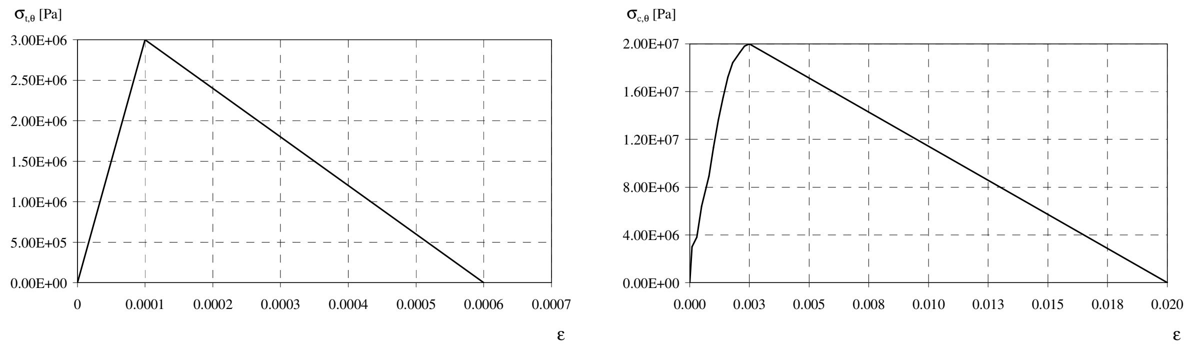


Fig. 5 – Stress – strain behaviour for steel S235.

Concrete is a quasi-brittle material with different behaviour in compression and in tension. Concrete is capable of directional integration point cracking and crushing besides incorporating plastic behaviour, see figure 6. 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 a specific failure surface. In this case, the plasticity is verified before the cracking and crushing checks.



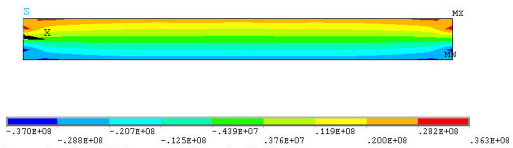
a) Stress-strain behaviour of concrete in tension. b) Stress-strain behaviour of concrete in compression.

Fig. 6 - Strength for concrete C20/25 material.

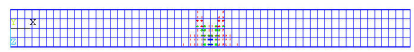
The presence of a crack failure mode at an integration point is represented through modification of the stress-strain relations by introducing a plane of weakness in a direction normal to the crack face. If stress relaxing is considered in tension, the secant modulus works with adaptive descent and diminishes to 0.0 as the solution converges, see figure 6. A multiplier coefficient of 1.0 was used for determining the amount of tensile stress relaxation. Shear transfer coefficients for open and closed cracks are also introduced ($\beta_t = 0.25$ and $\beta_c = 0.90$) representing a shear strength reduction factor for those subsequent loads, which induce sliding across the crack surface. 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.

NUMERICAL RESULTS

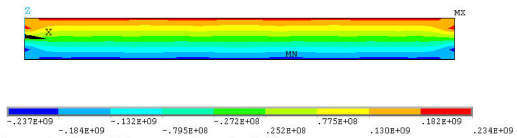
The real behaviour of partially encased steel beams is significantly different from the ideal. Buckling develops since the very beginning of the loading by equilibrium divergence. Steel and concrete follow a typical force - lateral displacement behaviour that is characteristics for this instability phenomenon (LTB). Steel will become actively yielding while concrete will failure by cracking and crushing. Figure 7 represents the evolution of yielding and failure of both materials.



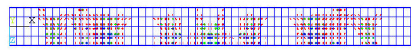
a) Steel: longitudinal stress for M=6136 [Nm].



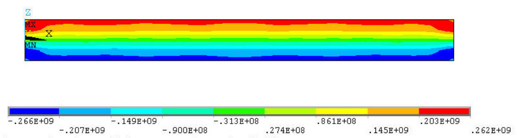
b) Concrete: cracking - crushing for M=6136 [Nm].



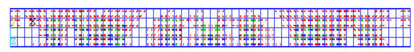
c) Steel: longitudinal stress for M=40413 [Nm].



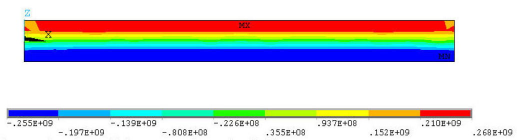
d) Concrete: cracking - crushing for M=40413 [Nm].



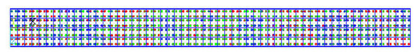
e) Steel: longitudinal stress for M=51950 [Nm].



f) Concrete: cracking - crushing for M=51950 [Nm].



g) Steel: longitudinal stress for M=57120 [Nm].



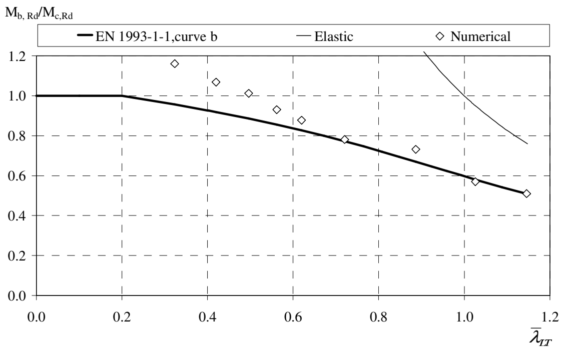
h) Concrete: cracking - crushing for M=57120 [Nm].

Fig. 7 – Lateral torsional buckling behaviour for partially encased beam with 2 [m] length, web plane.

The ultimate moment resistance will be determined for the last bending moment increment for which it is possible to sustain the equilibrium. Figure 7 also represents the results of the smeared approach for cracking and crushing failures, revealing progressive damage, reducing load-bearing capacity. Cracking is represented with a circle outline in the plane of the crack, and crushing is shown with an octahedron outline. If the crack has opened and then closed, the circle outline will have an X through it. Each integration point can crack in up to three different planes. The first crack at an integration point is shown with a red circle outline, the second crack with a green outline, and the third crack with a blue outline.
The ultimate LTB moment resistance corresponds to the maximum load-bearing capacity, being the non-dimensional slenderness the governing parameter. The larger slenderness of a beam corresponds to a smaller ultimate LTB resistance.

The reduction factor $\chi_{LT} = M_{b,Rd} / M_{c,Rd}$ should be derived from a specific buckling curve. No allowance for LTB will be considered for non-dimensional slenderness smaller than $\bar{\lambda}_{LT,0} = 0.2$.

Figure 8 represents the design buckling resistance for partially encased steel 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.



$\bar{\lambda}_{LT}$	$M_{b,Rd} / M_{c,Rd}$
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. 8 – Lateral torsional buckling design for partially encased steel with concrete.

CONCLUSIONS

Partially encased steel beams present grater flexural and torsional resistance when compared to similar steel rolled beam. Load-bearing capacity was presented for this type of composite beam, for different beam lengths, comparing the numerical results with the simple design calculation method presented in Eurocode (CEN, 2004-2005).

A three-dimensional model for partially encased beams was presented based on solid, shell and non-linear spring finite element for reproducing bond slip conditions.

Non-linear geometric and material finite element analysis was used for determining LTB resistance for partially encased beams without reinforcement.

For reduced values of the non-dimensional slenderness ($\bar{\lambda}_{LT} < 0.6$) the numerical results differ from the results presented by the simple design formula of Eurocode.

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