Load carrying capacity of partially encased columns for different fire ratings

Abdelkadir Fellouh,[1] Nourredine Benlakehal,[1] Paulo Piloto,[2] Ana Ramos,[3] Luís Mesquita[2]

[1]University Hassiba Benboual, Chlef, Algeria; [2]Polytechnic Institute of Bragança, Bragança, Portugal; [3]University of Salamanca, Salamanca, Spain

Abstract

Partially encased columns have significant fire resistance in comparison with steel bare columns. However, it is not possible to assess the fire resistance of such members simply by considering the temperature of the steel. The presence of concrete increases the mass and thermal inertia of the member and the variation of temperature within the cross section, in both the steel and concrete components. The annex G of EN1994-1-2:2005[1] allows to calculate the load carrying capacity of partially encased columns, for a specific fire rating time, considering the balanced summation model. New formulas will be proposed to calculate the plastic resistance to axial compression and the effective flexural stiffness. These two parameters are used to determine the buckling resistance. The finite element method is used to compare the results for the elastic critical load and the load carrying capacity of partially encased columns for different fire ratings of 30 and 60 min. This work compares the results from both solution methods, provides the validation of the three-dimensional model and demonstrates that a new design curve should be used for the buckling analysis of partially encased columns.

Introduction

Partially encased columns are usually made of hot rolled steel profiles, reinforced with concrete between the flanges. The composite section is responsible for increasing the torsional and bending stiffness when compared to the same section of the steel profile. In addition to these advantages, the reinforced concrete is responsible for increasing the fire resistance. Two methods are used to compare the member elastic buckling resistance and the axial load carrying capacity: the simple calculation method and the advanced calculation method. Two types of cross section were selected to study the effect of fire: IPE ranging from 200 to 500 and HEB ranging from 160 to 500. The columns were tested under fire ISO834:1999.[1] The critical load has been compared for columns with 3 and 5 m, with three different boundary conditions (pinned end at extremities, fixed end at extremities and mixed at extremities). Properties for steel were assumed from S275 grade and B500 grade, while C20/25 was assumed for concrete. The cross sections were defined accordingly to the tabulated data to design partially encased columns under fire.[2] This leads to minimum dimensions and minimum distances between components. The design of this section depends on the load level, and on the ratio between the thickness of the web and the thickness of the flange, see Figure 1. This tabulated data applies to structural several steel grades S235, S275 and S355 and to a minimum value of reinforcement, between 1 and 6%. The tabulated data uses values defined for the most common cross-sections based on experimental and empirical results. These results are generally very conservative and may be used for a preliminary design stage.[3] The simplified calculation method was originally developed by Jungbluth (1982),[4] and was defined to determine the capacity of the partially encased columns by dividing the section into four components (Figure 1). The current approach of this method is defined in Eurocode 4 part 1.2[2] and is based on simple formulas and empirical coefficients that seem to be unsafe.[5] For this purpose, a new simple formulae was presented and validated.[6] Table 1 presents the main dimensions, in particular the number of reinforcing bars, the diameter of each bar, the cover dimensions in both principal directions.

Materials and Methods

Fire design of partially encased columns

Eurocode 4 part 1-2(2005),[2] allows different methods to determine the fire resistance of partially encased columns under standard fire ISO834(1999).[1] The balanced summation model uses four components (flange, web concrete and reinforcement). The stability of partially encased columns requires the calculation of the critical load and the effective flexural stiffness. These quantities depend on the temperature effect on the elastic modulus and on the second order moment of area of each component, see eq. 1.

$$(EI)_{fi,eff,z} = \varphi_{f,\theta}(EI)_{fi,f,z} + \varphi_{w,\theta}(EI)_{fi,w,z} + \varphi_{c,\theta}(EI)_{fi,c,z} + \varphi_{s,\theta}(EI)_{fi,s,z} \quad (1)$$

Correspondence: Paulo Piloto, Polytechnic Institute of Bragança, Campus Santa Apolónia, 5300253 Bragança, Portugal.
Tel: +351.967458205.
E-mail: ppiloto@ipb.pt

Key words: Partially encased columns; Stability; Fire resistance; Balanced summation model.

Received for publication: 30 July 2016.
Revision received: 11 December 2016
Accepted for publication: 12 December 2016.

Fire Research 2017; 1:23
doi:10.4081/fire.2017.23

In this equation $(EI)_{fi,eff,z}$ represents the effective flexural stiffness of the composite section in fire, $(EI)_{fi,f,z}$ represents effective flexural stiffness of the flange, $(EI)_{fi,w,z}$ represents effective flexural stiffness of the web, $(EI)_{fi,c,z}$ represents the effective flexural stiffness of the concrete and $(EI)_{fi,s,z}$ represents the effective flexural stiffness of reinforcement. The contribution of each part is going to be weighted according to factors (Table 2).

The elastic buckling load $(N)_{fi,cr,z}$ requires the calculation of the axial plastic resistance under fire $(N)_{fi,pl,Rd}$. The non-dimensional slenderness ratio λ_θ is also calculated according to eqs. 2-4, when the safety partial factors are assumed equal to 1.0. The buckling length of the column under fire conditions is represented by L_θ.

$$N_{fi,pl,Rd} = N_{fi,pl,Rd,f} + N_{fi,pl,Rd,w} + N_{fi,pl,Rd,c} + N_{fi,pl,Rd,s} \quad (2)$$

$$\bar{\lambda}_\theta = \sqrt{N_{fi,pl,Rd} / N_{fi,cr,z}} \quad (3)$$

$$N_{fi,cr,z} = \pi^2 / L_\theta^2 \times (EI)_{fi,eff,z} \quad (4)$$

This calculation depends on the design value of the plastic resistance of each component: $(N)_{fi,pl,Rd,f}$ to the flanges, $(N)_{fi,pl,Rd,w}$ to the web, $(N)_{fi,pl,Rd,c}$ to the concrete and $(N)_{fi,pl,Rd,s}$ to the reinforced bars.

New proposal for the balanced summation model

This new proposal arises from the 2D thermal analysis of the cross section, taking into consideration certain hypotheses.[5] The new proposal to be used in the balanced summation model requires analytical formulas to take into consideration the effect of the fire in four components, assuming the same methodology of EN1994-1-2 annex G.[2] The average temperature of the flange is affecting the mechanical properties of the material without reduction of the second order moment of area and area of the flanges. The temperature is affecting part of the web to be neglected and consequently the second order moment of area and area of the web is modified, without considering any reduction of the mechanical properties. Temperature is also affecting the residual area and average temperature of the concrete, modifying the second order moment of area and area of the concrete, including also the mechanical properties of the concrete. The last component is also affected by and the average temperature of reinforcing bars, affecting only the mechanical properties of steel.

The flange component requires a bilinear approximation for the calculation of the average temperature instead of the linear approximation currently proposed in EN1994-1-2, using a new empirical coefficient k_t and a new reference value $\theta_{0,t}$, see eq. 5 and Table 3. This new proposal differentiates the type of the profile to be used in the partially encased column (HEB and IPE).

$$\theta_{f,t} = \theta_{0,t} + k_t\left(A_m/V\right) \quad (5)$$

The effect of fire on the web of the steel section is determined by the 400°C isothermal criterion.[7-9] This procedure defines the affected zone of the web and predicts the web height reduction $h_{w,fi}$ (Figure 1). These new formulae[5] presents a strong dependence on the section factor, regardless of the fire resistance class, unlike the simplified method of EN1994-1-2.[1] The results of the current version of Annex G in EN1994-1-2 are unsafe for all fire resistance classes and for all section factors. The new proposal[5,6] presents a parametric expression that depends on section factor and standard fire resistance class, eqs. 6-7. Both equations have the application limits defined in Table 4 and should be applied in accordance to the profile to be used.

$$2h_{w,fi}/h_i \times 100 = 0.0035 \times t^2 \times \left(A_m/V\right) - 0.03 \times t^{2.02} + \left(A_m/V\right)/2 \quad ,(HEB) \quad (6)$$

$$2h_{w,fi}/h_i \times 100 = 0.002 \times t^2 \times \left(A_m/V\right) - 0.03 \times t^{1.933} + \left(A_m/V\right) \quad ,(IPE) \quad (7)$$

The effect of the fire on the concrete was determined by the 500°C isothermal. The external layer of concrete to be neglected may be calculated in both principal directions, defining $b_{c,fi,v}$ and $b_{c,fi,h}$. According to EN1994 part 1.2,[2] the thickness of concrete to be neglected depends on section factor A_m/V, for standard fire resistance classes of R90 and R120. The new proposal[5,6] demonstrates a strong dependence on the section factor for all standard fire resistance classes, see eq. 8, and the applications conditions defined in Tables 5 and 6. The new proposal also differentiates the layer of concrete to be neglected in both principal directions. The average temperature of the residual concrete section may be calculated according to eqs. 9-10. The new proposal introduces a parametric approximation, based on the standard fire resistance t and section factor A_m/V. The application limits are presented in Table 7.

$$b_{c,fi} = a \times \left(A_m/V\right)^2 + b \times \left(A_m/V\right) + c \quad (8)$$

$$\theta_{c,t} = +3.1 \times t^{0.5} \times \left(A_m/V\right) + 0.003 \times t^{1.95} \quad ,(HEB) \quad (9)$$

Table 1. Section properties for partially encased columns.

Profile	Bars (n)	h_i	Φ (mm)	A_s (mm)	A_c (mm²)	u_1 (mm²)	u_2 (mm)	u (mm)	$\frac{A_s}{A_s + A_c}$	$\frac{t_w}{t_f}$	A_m/V (m⁻¹)
HEB160	4	134.0	12	452	19916	40	40	40	2.22	0.62	25.00
HEB180	4	152.0	12	452	25616	40	40	40	1.74	0.61	22.22
HEB200	4	170.0	20	1257	31213	50	50	50	3.87	0.60	20.00
HEB220	4	188.0	25	1963	37611	50	50	50	4.96	0.59	18.18
HEB240	4	206.0	25	1963	45417	50	50	50	4.14	0.59	16.67
HEB260	4	225.0	32	3217	53033	50	50	50	5.72	0.57	15.38
HEB280	4	244.0	32	3217	62541	50	50	50	4.89	0.58	14.29
HEB300	4	262.0	32	3217	72501	50	50	50	4.25	0.58	13.33
HEB320	4	279.0	32	3217	77275	50	50	50	4.00	0.56	12.92
HEB340	4	297.0	40	5027	80509	50	50	50	5.88	0.56	12.55
HEB360	4	315.0	40	5027	85536	50	50	50	5.55	0.56	12.22
HEB400	4	352.0	40	5027	95821	70	50	59	4.98	0.56	11.67
HEB450	4	398.0	40	5027	108801	70	50	59	4.42	0.54	11.11
HEB500	4	444.0	40	5027	121735	70	50	59	3.97	0.52	10.67
IPE200	4	183.0	12	452	16823	50	40	45	2.62	0.66	30.00
IPE220	4	201.6	20	1257	19730	50	40	45	5.99	0.64	27.27
IPE240	4	220.4	20	1257	23825	50	40	45	5.01	0.63	25.00
IPE270	4	249.6	25	1963	30085	50	40	45	6.13	0.65	22.22
IPE300	4	278.6	25	1963	37848	50	40	45	4.93	0.66	20.00
IPE330	4	307.0	25	1963	44854	50	40	45	4.19	0.65	18.56
IPE360	4	334.6	32	3217	50988	50	40	45	5.93	0.63	17.32
IPE400	4	373.0	32	3217	60715	70	40	53	5.03	0.64	16.11
IPE450	4	420.8	32	3217	72779	70	40	53	4.23	0.64	14.97
IPE500	4	468.0	40	5027	83800	70	50	59	5.66	0.64	14.00

$$\theta_{c,t} = +2.67 \times t^{0.5} \times (A_m/V) + 3.4 \times t^{0.61}, (IPE) \quad (10)$$

The effect of the fire into the reinforcing bars depends on the calculation of the average temperature of the material. The new parametric formula may be used to determine this effect. Eqs. 11-12 were developed, based on the distance between the reinforcing bars and the geometric averaged exposed surface u, fire resistance class t and section factor A_m/V.

$$\theta_{s,t} = 0.1 \times t^{1.1} \times (A_m/V) + 7.5 \times t - 0.1 \times t^{1.765} - 8 \times u + 390 \quad ,(HEB) \quad (11)$$

$$\theta_{s,t} = 14.0 \times (A_m/V) + 11.0 \times t - 0.1 \times t^{1.795} - 8 \times u + 115 \quad ,(IPE) \quad (12)$$

Advanced calculation method

The advanced calculation method is based on finite element analysis. A four step, uncoupled thermal and mechanical analysis is required to determine the buckling resistance of partially encased columns. The first step should be a nonlinear thermal analysis, the second step should be the elastic buckling analysis, the third step should be the nonlinear geometric and material plastic analysis and the fourth step is the nonlinear geometric and material buckling analysis. The 3D mesh was applied to the PEC after a convergence test of the solution using different sizes in Z direction. 50 element divisions were selected for columns with 3 m and 80 element divisions for columns with 5 m. The size of the mesh applied to the cross section was based on a previous experience of the simulation for 2D analysis.[5,6]

Nonlinear transient thermal analysis

The first step considers the nonlinear transient thermal analysis to calculate the temperature field. The finite element method requires the solution of eq. 13 in the internal domain of the partially encased column and eq. 14 in the external surface, when exposed to fire. In these equations: T represents the temperature of each material; $\rho(T)$ defines the specific mass; $C_p(T)$ defines the specific heat; $\lambda(T)$ defines the thermal conductivity; α_c specifies the convection coefficient; T_g represents the gas temperature of the fire compartment, using standard fire ISO 834[2] around the cross section (4 exposed sides); specifies the view factor; ε_m represents the emissivity of each material; ε_f specifies the emissivity of the fire; σ represents the Stefan-Boltzmann constant.

$$\nabla \cdot (\lambda_{(T)} \cdot \nabla T) = \rho_{(T)} \cdot C_{p(T)} \cdot \partial T/\partial t \quad (\Omega) \quad (13)$$

$$(\lambda_{(T)} \cdot \nabla T) \cdot \vec{n} = \alpha_c (T_g - T) + \Phi \cdot \varepsilon_m \varepsilon_f \cdot \sigma \cdot (T_g^4 - T^4) \quad (\partial\Omega) \quad (14)$$

The three-dimensional model uses element SOLID70 and element LINK33 to model the profile/concrete and rebars, respectively. SOLID70 has a 3-D thermal conduction capability, presents eight nodes with a single degree of freedom (temperature at each node). The interpolating functions are linear and this element uses full integration points (2x2x2) to define the conductivity matrix. The finite element LINK33 is a uniaxial element with the ability to conduct heat between two nodes. The element has a single degree of freedom, temperature at each node. The interpolating functions are linear and this element uses exact integration to define the conductivity matrix. Figure 2 represents the shape of each element. Perfect contact between reinforcing bars and concrete is assumed, being the nodes of both elements shared in space. The nonlinear transient thermal analysis was defined with an integration time step of 60 s, which can decrease to 1 s and increase up to 120 s. The criterion for convergence uses a tolerance value of the heat flow, smaller than 0.1% with a minimum reference value of 1x10^{-6}. The thermal properties of concrete follow the models defined in EN1992-1-2,[10] assuming 3% of moisture contents by weight of siliceous concrete and the upper limit for the conductivity, according to the recommendation of EN1994-1-2.[2] The thermal properties of steel were defined according to the models defined on EN1993-1-2.[11]

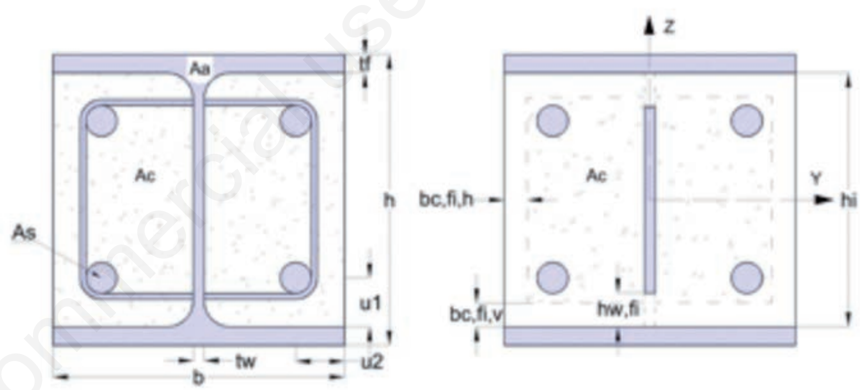

Figure 1. Balanced summation model for partially encased columns under fire.

Table 2. Reduction coefficients for bending stiffness around the week axis.

Standard fire resistance	$\varphi_{f,\theta}$	$\varphi_{w,\theta}$	$\varphi_{c,\theta}$	$\varphi_{s,\theta}$
R30	1.0	1.0	0.8	1.0
R60	0.9	1.0	0.8	0.9
R90	0.8	1.0	0.8	0.8
R120	1.0	1.0	0.8	1.0

Table 3. Parameters used for the calculation of flange temperature (section HEB and IPE).

Sections	10<A_m/V<14 HEB		14<=A_m/V<25 HEB		10<A_m/V<19 IPE		19<=A_m/V<30 IPE	
Standard fire	$\theta_{0,t}$ (°C)	k_t (m°C)	$\theta_{0,t}$ (°C)	k_t (m°C)	$\theta_{0,t}$ (°C)	k_t (m°C)	$\theta_{0,t}$ (°C)	k_t (m°C)
R30	387	19.55	588	4.69	582	6.45	656	2.45
R60	665	14.93	819	3.54	824	3.75	862	1.72
R90	887	5.67	936	2.04	935	2.20	956	1.09
R120	961	4.29	998	1.62	997	1.68	1010	0.96

A total of 48 simulations were developed to account for the temperature of 24 different cross sections and 2 columns lengths (3 and 5 m). The temperature field was determined for the total time of 7200 s. Figure 3 shows an example of the temperature field for two examples of partially encased columns exposed to ISO834 fire, after 30 and 60 minutes. The temperature field was recorded for the corresponding resistance class and applied as body load to the mechanical model defined in next steps.

Static and eigen buckling analysis

The three-dimensional model uses element SOLID 185 to model the hot rolled steel, SOLID 65 to model concrete and LINK180 to model the reinforcing bars. SOLID 185 has eight nodes with three degrees of freedom at each node (displacements) and uses linear interpolating functions. The reduced integration method (Gauss point) was applied taking into consideration the comparison of the critical load with the analytical method. SOLID65 was elected to model concrete, presents eight nodes with three degrees of freedom at each node (displacements) and uses linear interpolating functions with full integration scheme (2x2x2 Gauss point). LINK180 was selected to model the reinforcing bars, using two nodes with three degrees of freedom (displacements), using linear interpolating functions with 1 Gauss point for integration scheme. Perfect contact between the reinforcing bars and concrete is assumed with sharing nodes.

This second step considers a static and eigen buckling analysis with Block Lanczos extraction method. The static linear analysis is the basis for the eigen buckling analysis. The solution of eq. 15 must be found primarily, assuming $\{F_{ref}\}$ is an arbitrary load on the partially encased column (usually a unit force). $[K]$ is its stiffness matrix and $\{d\}$ is the displacement vector. When the displacements are known, the stress field can be calculated for the reference load $\{F_{ref}\}$, which can be used to form the stress stiffness matrix $[K_{,ref}]$. Since the stress stiffness matrix is proportional to the load vector $\{F_{ref}\}$, an arbitrary stress stiffness matrix $[K]$ and an arbitrary load vector $\{F\}$ may be defined by a constant λ as shown by eqs. 16-17. The stiffness matrix is not changed by the applied load because the solution is linear. A relation between the stiffness matrices, the displacement and the critical load can then be presented as in eqs. 18-19, which can be used to predict the bifurcation point. The critical load is defined as $\{F_{cri}\}$. Since the buckling mode is defined as a change in displacement for the same load, eqs. 19-20 are still valid, where $\{\delta d\}$ represents the incremental buckling displacement vector. The difference between eq. 19 and eq. 20 produces an eigenvalue problem, represented by eq. 21 where the smallest root λ defines the first buckling load λ_{cri}, when bifurcation is expected. The mechanical properties of concrete follow the models defined in EN1994-1-2,[2] for normal weight concrete, with the assumption of elastic material under compression and tension. The mechanical properties of the reinforcing steel follow the model of EN1994-1-2,[2] and the mechanical properties of hot rolled steel follow the models of EN1993-1-2,[11] both assuming the elastic behaviour of materials.

Figure 2. Finite elements used to build the three-dimensional model of partially encased columns.

Table 4. Application limits for HEB and IPE cross sections regarding web component.

Standard fire resistance	Section factor (HEB)	Section factor (IPE)
R30	$A_m/V<22.22$	$A_m/V<30.00$
R60	$A_m/V<15.38$	$A_m/V<18.56$
R90	$A_m/V<12.22$	$A_m/V<14.97$
R120	$A_m/V<11.11$	-

Table 5. Parameters and application limits for thickness reduction of the concrete in sections HEB.

Standard fire resistance	$b_{c.fi.h}$ a	b	c	$b_{c.fi.v}$ a	b	c	Section factor
R30	0.0000	0.0809	13.5	0.000	0.372	3.5	$10<=A_m/V<=25$
R60	0.1825	-4.2903	50.0	0.1624	-3.2923	41.0	$10<=A_m/V<=20$
R90	1.0052	-22.575	163.5	1.8649	-43.287	298.0	$10<=A_m/V<=17$
R120	0.0000	7.5529	-35.5	0.000	6.0049	9.0	$10<=A_m/V<=13$

Table 6. Parameters and application limits for thickness reduction of the concrete in sections IPE.

Standard fire resistance	$b_{c.fi.h}$ a	b	c	$b_{c.fi.v}$ a	b	c	Section factor
R30	0.0000	0.2206	10.5	0.0000	0.9383	-3.0	$14<=A_m/V<=30$
R60	0.2984	-8.8924	93.0	0.5888	-15.116	135.0	$14<=A_m/V<=22$
R90	1.3897	-38.972	313.0	2.0403	-50.693	393.0	$14<=A_m/V<=17$
R120	0.0000	18.283	-199.0	0.0000	48.59	-537.0	$14<=A_m/V<=15$

$$[K]\{d\}=\{F_{ref}\} \tag{15}$$

$$[K_{\sigma}]=\lambda[K_{\sigma,ref}] \tag{16}$$

$$\{F\}=\lambda\{F_{ref}\} \tag{17}$$

$$\{F_{cri}\}=\lambda_{cri}\{F_{ref}\} \tag{18}$$

$$\left[[K]+\lambda_{cri}\left[K_{\sigma,ref}\right]\right]\{d\}=\lambda_{cri}\{F_{ref}\} \tag{19}$$

$$\left[[K]+\lambda_{cri}[K_{\sigma,ref}]\right][\{d\}+\{\delta d\}]=\lambda_{cri}\{F_{ref}\} \tag{20}$$

$$\left[[K]+\lambda\,[K_{\sigma,ref}]\right]\{\delta d\}=\{0\} \tag{21}$$

A total of 288 simulations were developed, taking into consideration 24 different cross sections, 2 columns lengths, 3 buckling lengths and 2 fire ratings. The trivial solution is not of interest, which means that the solution for λ is defined for an algebraic equation, imposing the determinant of the global matrix equal to zero. The calculated eigenvalue is always related to an eigenvector $\{\delta d\}$ called a buckling mode shape (Figure 4). This numerical solution of a linear buckling analysis assumes that everything is perfect and therefore the real buckling load will be lower than the calculated buckling load if the imperfections are taking into account.

Plastic resistance analysis

The plastic resistance was also evaluated, taking into consideration the criterion for the plastic behaviour of the column (Figure 5). A perfect geometry was elected for every type of cross section (24) and for two fire ratings (30 and 60 minutes), making a total of 48 3D simulations. The elements were prevented to move laterally and the bottom of the column was fixed to the ground. A compressive displacement was applied on the top of the column with a typical incremental displacement of 0.1 mm, with possibility to decrease up to 0.01 mm and to increase up to 0.2 mm. The iterative and incremental simulation is material and geometric nonlinear, and used the criterion for convergence based on displacement, with a tolerance value of 5%. The mechanical properties of concrete follow the models defined in EN1994-1-2,[2] for normal weight concrete, with the assumption of elastic – perfectly plastic material under compression and tension. The mechanical properties of the reinforcing steel follow the model of EN1994-1-2,[2] and the mechanical properties of hot rolled steel follow the models of EN1993-1-2,[11] both assuming the elastic – perfectly plastic behaviour of materials.

Similar element types and meshes were used for this simulation, being the reinforcing bars the last component to become plastic. The plastic resistance was defined by the reaction force.

This calculation was performed to allow for the calculation of the non-dimensional slenderness ratio, using the numerical results for each specified fire rating, λ_{θ}, taking also into consideration the results of the previous section.

Nonlinear buckling resistance analysis

Eigenvalue buckling analysis predicts the theoretical buckling strength (the bifurcation point) of an ideal linear elastic structure. However, imperfections and nonlinearities prevent most real-world structures from achieving their theoretical elastic buckling strength. The nonlinear buckling analysis is a static analysis with large deflection (equilibrium in deformed configuration), extended to a point where the structure reaches its ultimate limit state (plasticity, modification into a mechanism). The mechanical properties of materials follow the same models used in the previous section. The buckling load is the maximum load determined from the curve plotted for load displacement (Figure 6).

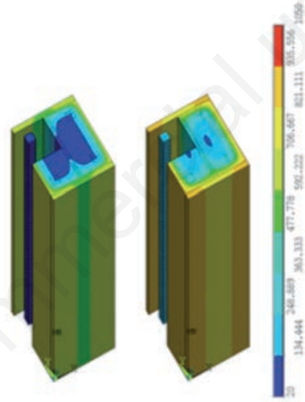

Figure 3. Temperature results for partially encased column HEB 360 for 30 and 60 minutes.

Figure 4. Buckling modes after 30 (IPE330, left) and 60 minutes (HEB160, right) of fire.

Table 7. Application limits for average temperature of the concrete.

Standard fire resistance	Section factor (HEB)	Section factor (IPE)
R30	$A_m/V<25$	$A_m/V<30$
R60	$A_m/V<20$	$A_m/V<23$
R90	$A_m/V<17$	$A_m/V<18$
R120	$A_m/V<14$	$A_m/V<15$

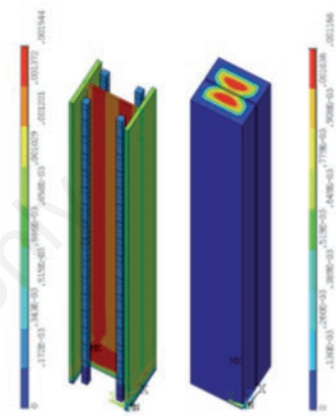

Figure 5. Plastic strain after 30 minutes of fire (HEB 360).

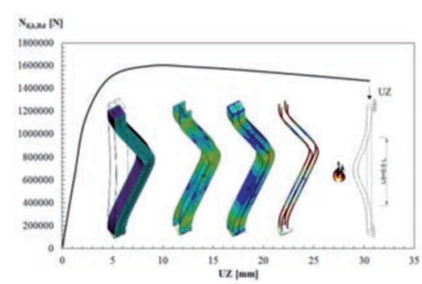

Figure 6. Buckling mode and resistance of HEB160 for 30 minutes of fire and fixed in both ends.

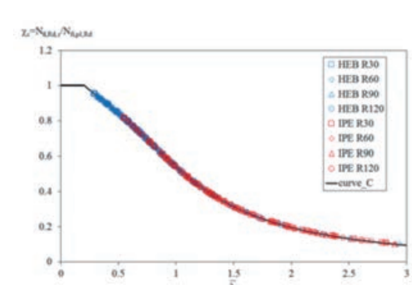

Figure 7. Buckling resistance of HEB160 for 30 minutes of fire and fixed in both ends.

Table 8. Buckling resistance for PEC, under fixed end conditions.

Profile	A_m/V	$N_{fi.b.Rd}$ (ANSYS) (n) 3 m R30	3 m R60	5 m R30	5 m R60
HEB160	25.00	1625700	1601100	1207800	1132900
HEB180	22.22	2093500	2019900	1618700	1518200
HEB200	20.00	2840400	2779000	2298100	2180700
HEB220	18.18	3645000	3583900	3055000	2924700
HEB240	16.67	4213300	4154700	3622400	3468000
HEB260	15.38	5252500	5195900	4639400	3707100
HEB280	14.29	5823500	NB	5260100	4489700
HEB300	13.33	NB	NB	5865400	5094800
HEB320	12.92	NB	NB	6257400	6092700
HEB340	12.55	NB	NB	7336000	7189300
HEB360	12.22	NB	NB	7668600	7522200
HEB400	11.67	NB	NB	8565500	8210400
HEB450	11.11	NB	NB	8904600	8747400
HEB500	10.67	NB	NB	9613900	9442600
IPE200	30.00	764090	682200	355810	328890
IPE220	27.27	1198900	1033100	542680	497550
IPE240	25.00	1408200	1321000	733740	671740
IPE270	22.22	2001400	1919800	1167100	1069500
IPE300	20.00	2444700	2369500	1612000	1487000
IPE330	18.56	2822000	2747600	1982500	1825400
IPE360	17.32	3709200	3630400	2768700	2592800
IPE400	16.11	4213200	4200000	3559800	3061100
IPE450	14.97	4794300	4759800	3811900	3588700
IPE500	14.00	6283600	6454100	4122300	4863900

The buckling resistance of each column was calculated by the incremental displacement and iterative solution model, using Newton-Raphson method. The imperfection of the geometry was based on the elastic buckling mode shape from 2nd step, with an updating of the node coordinates. This update was based on the mode shape and based on the maximum imperfection expected on the mid high of the column, corresponding to L/150. Typical incremental displacement of 0.1 mm was applied, with minimum incremental displacement of 0.01 mm and maximum incremental displacement of 0.2 mm. The criterion for convergence was based on displacement with tolerance value of 5%.

A total of 96 simulations were performed for the case of 24 different cross sections, two column lengths (3 and 5 m) using fixed end supports (buckling length equal to 0.5 L) and for two fire resistance classes (30 and 60 minutes). Table 8 presents the results for the buckling load. The cells identified by NB means that, under specific conditions (buckling length and fire rating), this partially encased column did not attain the buckling mode of instability as a potential failure mode.

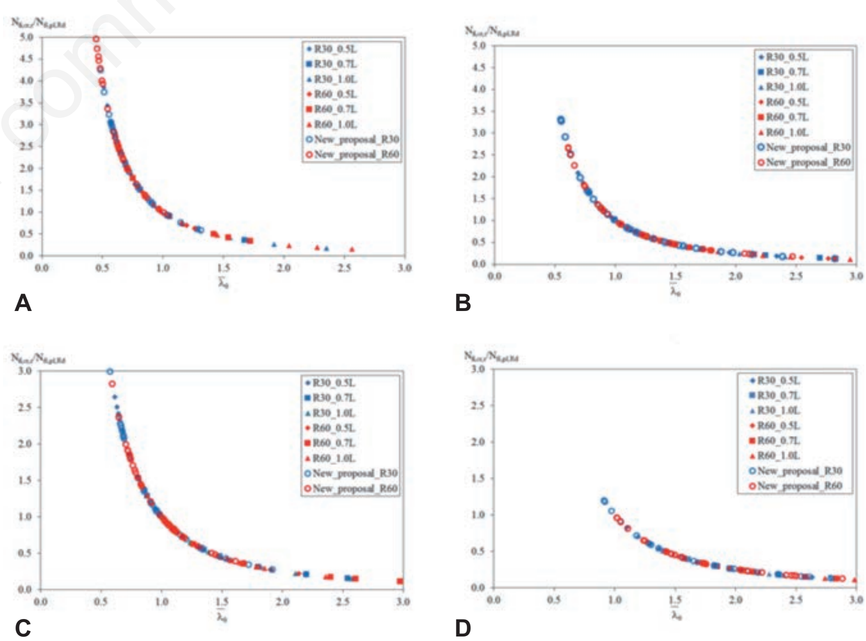

Figure 8. Ratio between critical and plastic resistance for 3 m of height and section HEB (A) and IPE (B), and for 5 m of height and section HEB (C) and IPE (D).

Results and Discussion

Results obtained from the balanced summation model

Figure 7 presents the buckling curve for partially encased columns, using the results from the new proposal used for the balanced summation model after 30, 60, 90 and 120 minutes of fire exposure and for different boundary conditions. The results where plotted using buckling curve C from EN1993-1-1.[12]

Results obtained from the eigen buckling analysis

Figure 8 presents the comparison of the buckling load, using the results from the new proposal and results from eigen buckling analysis, for 30 and 60 minutes of fire exposure and for different boundary conditions. The ratio between the critical load and the axial plastic resistance depends on the non-dimensional slenderness ratio and fits well with the new proposal used for the balanced summation model.

The numerical solution method is based on the elastic buckling analysis, considering the full resistance of the four components, taking into account the update of the material properties and the full geometry of column. This fact justifies that the numerical results are always higher that the ones presented by the new proposal. Partially encased columns from HEB profile present higher critical load when compared with IPE profile. The critical load decreases with the relative slenderness in the fire situation, as expected.

Results obtained from nonlinear buckling analysis

Figure 9 presents the comparison of the results obtained by nonlinear analysis (ANSYS) and the buckling curve proposed by EN1994-1-2,[2] when considering the buckling curve c from EN 1993-1-1.[12] The results are presented for 30 and 60 minutes of fire exposure and for the buckling length L_0=0.5 L. Partially encased columns submitted to 60 minutes of fire present smaller buckling resistance when compared with the same ones submitted to 30 minutes. These numerical results are based on the full resistance of the four components, taking into account the update of the material properties and the full geometry of column. This fact justifies that the numerical results are always higher that the ones presented by the EN1994-1-2.

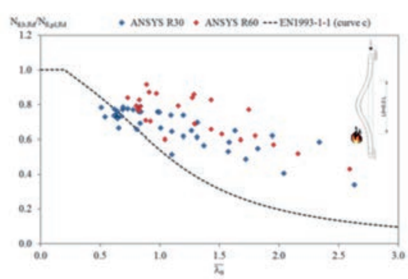

Figure 9. Buckling design curve (comparison between ANSYS and Eurocode).

Conclusions

The buckling analysis of partially encased column was analysed under fire conditions. Two different solution methods were applied to define the buckling resistance of such members. The simplified method proposed in Annex G EN1994-1-2,[2] is unsafe when compared to the numerical results.[5,6] The results of new proposal are based on the balanced summation method, as proposed in the current version of EN1994-1-2,[2] but using safer formulae. This new formulation is based on the evolution of the average temperature in the flange, based on the residual height of the web according to 400°C isothermal criterion, based on the residual concrete according to the 500°C isothermal criterion and its average temperature, and finally based on the average temperature of the reinforcing bars. The numerical solution method was defined as a 4-step procedure. The elastic buckling analysis, considers the full resistance of the four components, updating the material properties and the full geometry of columns. This fact justifies that the numerical results are always higher than the ones presented by the new formulae. According to the elastic buckling results, good agreement was found between the new proposal and the numerical simulation, concluding that the new proposal is safe. Partially encased column presents higher buckling resistance than bare steel columns. It was also verified that the buckling resistance decreases with the buckling length and for higher fire rating classes, smaller buckling loads are expected. The material and geometric non-linear analysis revealed that the buckling curve suggested by EN1994-1-2 is not safe and a different curve fit should be proposed. This study must be extended to other types of cross section and different configurations of PEC. Experimental tests are also required to validate the best curve to fit the results.

References

1. ISO 834-1. Fire-resistance tests. Elements of building construction. Part 1: general requirements. 1999. pp. 25.
2. CEN-EN 1994-1-2. Eurocode 4 - Design of composite steel and concrete structures- Part 1-2: general rules. Structural fire design. Brussels, August 2005, pp. 109.
3. Arezki S, Said I. Practical fire design of partially encased composite steel-concrete columns according to Eurocode 4. International Congress on Materials & Structural Stability, 2014. MATEC Web of Conferences 11, 01029-pp. 1-8.
4. Jungbluth O. Optimierte Verbandbauteile. Köeln: Stahlbau Verlags GmbH; 1982.
5. Piloto P, Almeida D, Ramos-Gavilán AB, Mesquita LMR. Partially encased section: strength and stiffness under fire conditions. In: Proceedings of the IFireSS-International Fire Safety Symposium. University of Coimbra, Portugal, 20th-22nd April 2015. pp. 15-8.
6. Piloto P, Mesquita L, Ramos-Gavilán AB, Almeida D. Axial buckling load of partially encased columns under fire. New formulae. Proceedings of the X Congress for steel and composite construction. Iparque Coimbra, 26-27 November 2015;2:383-92.
7. Cajot L-G, Gallois L, Debruyckere R, Franssen J-M. Simplified design method for slim floor beams exposed to fire. Proceedings of the Nordic Steel Construction Conference, Oslo, Norway, 5-7 September, 2012. pp. 1-11.
8. Zaharia R, Duma D, Vassart O, et al. Simplified method for temperature distribution in slim floor beams. Prague: Application of structural fire design; 2011.
9. Zaharia R, Franssen JM. Simple equations for the calculation of the temperature within the cross-section of slim floor beams under ISO. Fire, Steel Comp Struct 2012;13:171-85.
10. CEN-EN 1992-1-2. Eurocode 2: Design of concrete structures - Part 1-2: General rules structural fire design. Brussels: European standards; 2004.
11. CEN-EN 1993-1-2. Eurocode 3: Design of steel structures - Part 1-2: General rules - Structural fire design. Brussels: European standards; 2005.
12. CEN-EN 1993-1-1. Eurocode 3: Design of steel structures - Part 1-1: General rules - Rules for buildings. Brussels: European standards; 2005.