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PREFACE

This book contains the abstracts of the papers presented in the 7th International Conference on Safety and Durability of Structures (ICOSADOS 2016), held in the University of Trás-os-Montes e Alto Douro (UTAD), city of Vila Real, Portugal, from 10th to 12th of May 2016.

A contribution in the internationalisation goal of ICOSADOS was achieved with this event taking into account that authors or members of the Scientific Committee of eight countries collaborated. These countries are Poland, Latvia, Portugal, UK, Italy, Mexico, France and Brazil.

There was also a significant participation of the industry which sponsored the conference and gave an important contribution for its success. The Civil Engineering students of UTAD also gave a relevant help in the organization of this conference.

In this conference there were four lectures presented by keynote speakers who are international references in the topics of safety and durability of structures. These keynote speakers are Professors Pawel Sniady (Wrocław University of Environmental and Life Sciences, Poland), Ulvis Skadins (Latvia University of Agriculture, Latvia), Jitendra Agarwal (University of Bristol, United Kingdom) and António Arêde (Engineering Faculty of University of Porto, Portugal).

The conference scope includes a wide range of safety and durability of structures topics such as:
S1 - Degradation: diagnostics and evaluation methods
S2 - Structural, physical and material characterisation
S3 - Numerical modelling
S4 - Natural and man-made risks
S5 - Requirements and code provisions
S6 - Assessment, conservation, repair and strengthening
S7 - Case studies

The Editors are grateful to all authors, members of the scientific committee and other colleagues that make possible the publication of this book.

The Editors
Vila Real
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STABILITY OF PARTIALLY ENCASED COLUMNS UNDER FIRE

Abdelkadir Fellouh[1, a], Nourredine Benlakehal[1,b], Paulo Piloto[2,c], Ana Ramos[3,d], Luís Mesquita[4,e]

[1]UHBC, Hay Salem, National Road 19, 02000 Chlef, Algeria

[2]LAETA-INEGI, IPB, Campus Sta. Apolónia, ap. 1134, 5301-857 Bragança, Portugal

[3]USAL – EPSZ, Campus Viriato, Av. Cardenal Cisneros, 34, 49022- Zamora, Spain

[4]ISISE, IPB, Campus Sta. Apolónia, ap. 1134, 5301-857 Bragança, Portugal

[a]Redwan.epst@gmail.com, [b]n_blk@yahoo.fr, [c]ppiloto@ipb.pt, [d]aramos@usal.es, [c]lmesquita@ipb.pt

Keywords: partially encased columns, stability, fire resistance.

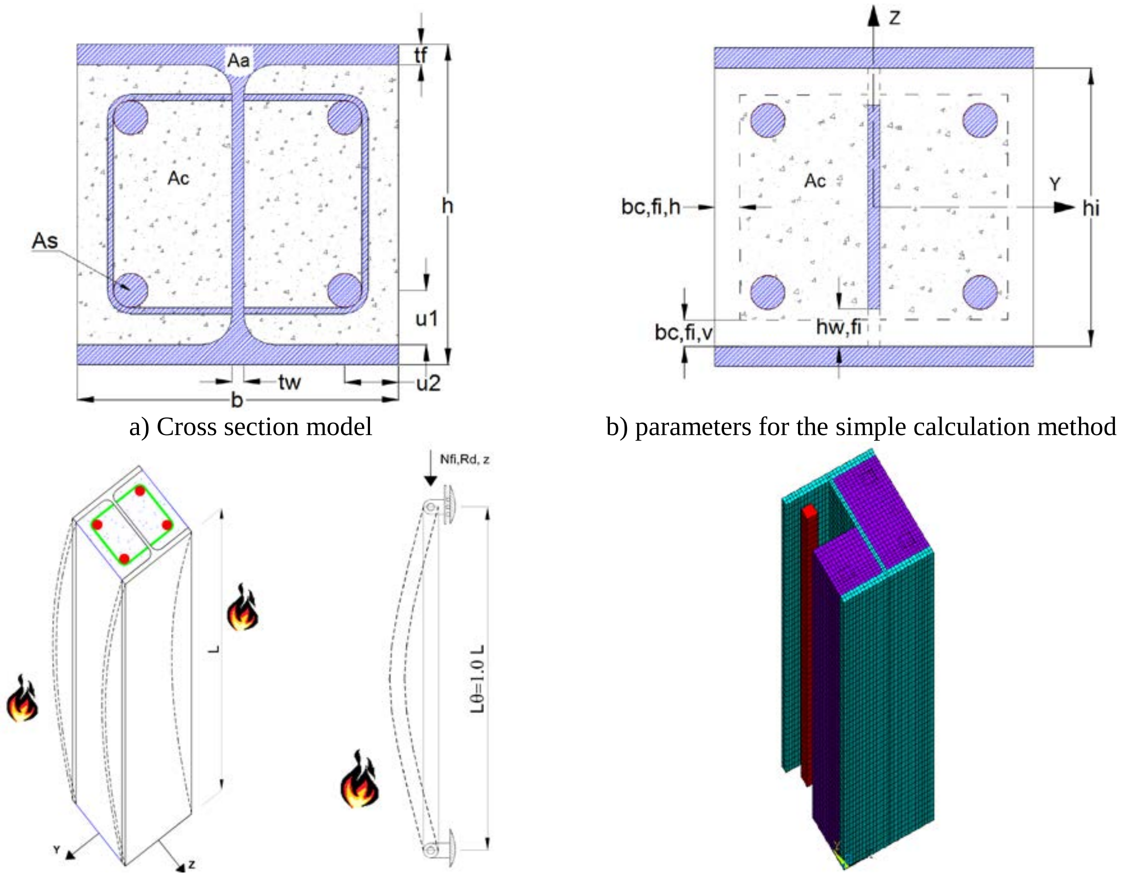
Introduction

The stability of partially encased columns under fire is evaluated, based on two different methods. The simple calculation method is presented and depends on new simple formulae, based on two major hypotheses, safer than the current method proposed in EN1994-1-2. This document establishes a designing method that considers the contours of temperature within the cross section after 30, 60, 90 and 120 minutes under fire exposure. The cross section is divided into four components in which the mechanical property of the material changes with the average temperature and part of the material is also neglected.
An advanced calculation method, fully three-dimensional, is used to compare the results of the axial critical load. The results agree very well for fire ratings of 30 and 60 minutes.

Partially Encased Columns

Partially encased columns are usually made of hot rolled steel profiles, reinforced with concrete between the flanges. The composite section increases the torsional and bending stiffness when compared to the same used bare steel column. The reinforced concrete is also responsible for increasing the fire resistance. The fire resistance of partially encased columns may be calculated by the balanced summation method, which is based on the contribution of four components and depends on the temperature effect in each component. According to Eurocode 4, Part 1.2 [1], the fire resistance can be evaluated by this method, considering the flanges of steel profile, the web of steel profile, the concrete and the reinforcement submitted to standard fire and for different fire resistance classes (R30, R60, R90 and R120). This paper aims to compare the results of the critical load, when calculated by the balanced summation method (new proposal, [2]) and the results of a fully three-dimensional model, herein presented.
Two types of cross section were selected to study the effect of fire, corresponding to a set of cross

section geometries: IPE ranging from 200 to 500 and HEB ranging from 160 to 500, see table 1. The critical load has been compared for columns with 3m, pinned in both ends. Fig. 1 identifies the four components, presents the finite element model and the expected buckling mode under fire. Properties for steel were assumed from S275 grade and B500 grade, while C20/25 was assumed for concrete.



a) Cross section model

b) parameters for the simple calculation method

c) buckling deformed shape mode and buckling length in fire

d) Finite element approximation for HEB360

Fig. 1: Partially encased column.

Table 1: Section properties for partially encased columns.

Profile	Rebars	h_i mm	Φ mm	A_s mm^2	A_c mm^2	u_1 mm	u_2 mm	u mm	A_s / $A_s + A_c$	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

Solution Methods
Two solution methods are to be used: The simple calculation method and the advanced calculation method. The simple calculation method is based on the analytical formulae of the new proposal [2] and the advance calculation method is based on a set of two finite element steps (nonlinear thermal analysis and elastic buckling analysis).

Simple calculation method. This method requires analytical formulas to take into consideration the effect of the fire [3] in four components, assuming the same methodology of EN1994-1-2 annex G [1]. For the calculation of the critical load, the effective flexural stiffness needs to be determined. This quantity depends on the temperature effect on the elastic modulus and on the second order moment of area of four components, according to Eq. 1. 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 φ factors, see table 2.

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

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

The elastic critical load $N_{fi,cr,z}$ follows Eq. 2, where L_θ represents the buckling length of the column under fire conditions.

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

The new proposal for the calculation of the effective stiffness of the flange, Eq. 3, is defined by a bilinear approximation for the average temperature calculation in flange, using a new empirical coefficient k_t and a new reference value $\theta_{0,t}$, see Table 3. The temperature is affecting the Elastic modulus of the material of this component without any other reduction of geometry to the second order moment of area.

$$\theta_{f,t} = \theta_{0,t} + k_t(A_m/V) \quad (3)$$

Table 3: Parameters to determine flange temperature (Section HEB and IPE).

Sections	10<Am/V<14		14<=Am/V<25		10<Am/V<19		19<=Am/V<30	
Standard	HEB		HEB		IPE		IPE	
Fire	$\theta_{0,t}$ [^{0}C]	k_t [m^{0}C]	$\theta_{0,t}$ [^{0}C]	k_t [m^{0}C]	$\theta_{0,t}$ [^{0}C]	k_t [m^{0}C]	$\theta_{0,t}$ [^{0}C]	k_t [m^{0}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

The effect of fire on the web of the cross section was determined by the 400 °C isothermal criterion [4-6]. This procedure defines the affected zone of the web and predicts the web height reduction

$h_{w,fi}$, see Fig. 1. This new formulae [2] presents a strong dependence on the section factor, regardless of the fire resistance class (t in minutes), unlike the simplified method of EN1994-1-2 [1]. The results of EN1994-1-2 [1] are unsafe for all fire resistance classes and for all section factors. The new proposal presents a parametric expression that depends on section factor and standard fire resistance class, Eqs. 4-5. Both equations have the application limits defined in Table 4. This calculation is affecting the second order moment of area of the web, without considering any temperature effect on the reduction of the elastic modulus.

$$2h_{w,fi}/h_i \times 100 = 0.0035 \times t^2 \times (A_m/V) - 0.03 \times t^{2.02} + (A_m/V)/2 \quad ,(HEB) \tag{4}$$

$$2h_{w,fi}/h_i \times 100 = 0.002 \times t^2 \times (A_m/V) - 0.03 \times t^{1.933} + (A_m/V) \quad ,(IPE) \tag{5}$$

Table 4: Application limits for HEB and IPE cross sections.

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

The effect of the fire on the concrete was determined by the 500 ºC isothermal [1]. 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 EN1994-1-2 [1], the thickness of concrete to be neglected depends on section factor, for standard fire resistance classes of R90 and R120. The new proposal [2] demonstrates a strong dependence on section factors for all standard fire resistance classes, see Eq. 6, considering the applications conditions in Tables 5-6. The new proposal also differentiates the layer of concrete to be neglected in both principal directions.

$$b_{c,fi} = a \times (A_m/V)^2 + b \times (A_m/V) + c \tag{6}$$

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

	$b_{c,fi,h}$			$b_{c,fi,v}$			
Standard fire resistance class	a	b	c	a	b	c	Section factor
R30	0,0000	0,0809	13,5	0,000	0,372	3,5	10<=Am/V<=25
R60	0,1825	-4,2903	50,0	0,1624	-3,2923	41,0	10<=Am/V<=20
R90	1,0052	-22,575	163,5	1,8649	-43,287	298,0	10<=Am/V<=17
R120	0,0000	7,5529	-35,5	0,000	6,0049	9,0	10<=Am/V<=13

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

	$b_{c,fi,h}$			$b_{c,fi,v}$			
Standard fire resistance class	a	b	c	a	b	c	Section factor
R30	0,0000	0,2206	10,5	0,0000	0,9383	-3,0	14<=Am/V<=30
R60	0,2984	-8,8924	93,0	0,5888	-15,116	135,0	14<=Am/V<=22
R90	1,3897	-38,972	313,0	2,0403	-50,693	393,0	14<=Am/V<=17
R120	0,0000	18,283	-199,0	0,0000	48,59	-537,0	14<=Am/V<=15

The average temperature of the residual concrete section may be calculated according to Eqs. 7-8. The new proposal introduces a parametric approximation, based on the standard fire resistance and section factor. The application limits are presented in Table 7. This calculation is affecting the second order moment of area of the concrete and also the elastic modulus.

$$\theta_{c,t} = +3.1 \times t^{0.5} \times (A_m/V) + 0.003 \times t^{1.95} \quad ,(HEB) \tag{7}$$

$$\theta_{c,t} = +2.67 \times t^{0.5} \times (A_m/V) + 3.4 \times t^{0.61} \quad ,(IPE) \tag{8}$$

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

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

The effect of the fire into the rebars depends on the calculation of the average temperature. The new parametric formula may be used to determine this effect. Eqs. 9-10 were developed, based on the distance between rebars exposed surface (u), fire resistance class (t) and section factor (Am/V).

$$\theta_{s,t} = 0.1 \times t^{1.1} \times (A_m/V) + 7.5 \times t - 0.1 \times t^{1.765} - 8 \times u + 390 \quad ,(HEB) \tag{9}$$

$$\theta_{s,t} = 14.0 \times (A_m/V) + 11.0 \times t - 0.1 \times t^{1.795} - 8 \times u + 115 \quad ,(IPE) \tag{10}$$

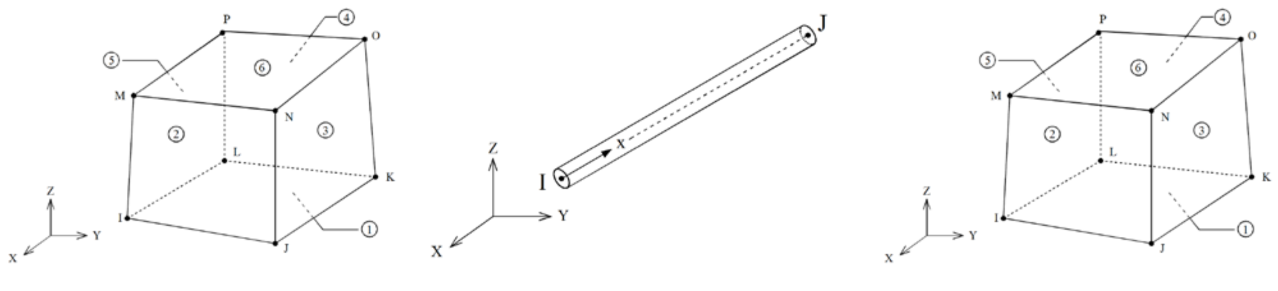
This calculation is affecting only the elastic modulus of the rebars.

Advanced calculation method. The nonlinear solution method (ANSYS) was used to evaluate the temperature field. The finite element method requires the solution of Eq. 11 in the internal domain of the partially encased column and Eq. 12 in the boundary. 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 curve ISO 834 [3] around the cross section (4 exposed sides); Φ 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.

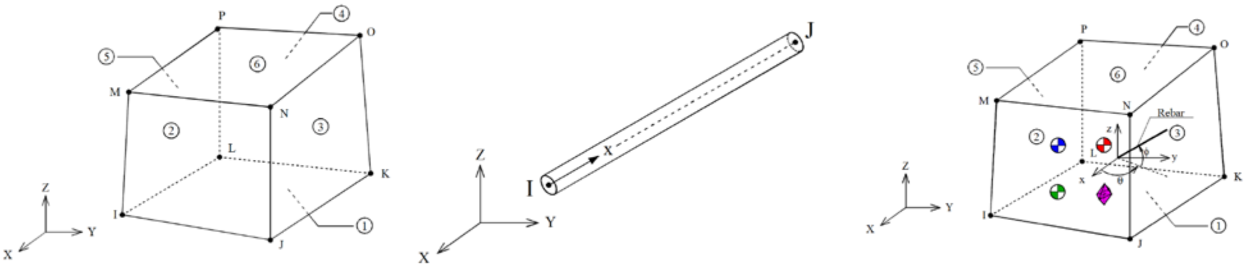
$$\nabla \cdot (\lambda_{(T)} \cdot \nabla T) = \rho_{(T)} \cdot C_{p(T)} \cdot \partial T / \partial t \qquad (\Omega) \tag{11}$$

$$(\lambda_{(T)} \cdot \nabla T) \cdot \vec{n} = \alpha_c (T_g - T) + \Phi \cdot \varepsilon_m \varepsilon_f \cdot \sigma \cdot (T_g^4 - T^4) \qquad (\partial\Omega) \tag{12}$$

The temperature field was determined by the finite element method, using ANSYS. The three dimensional element SOLID70 and the LINK33 were used to model the profile / concrete and rebars, respectively. SOLID70 has a 3-D thermal conduction capability, presenting eight nodes with a single degree of freedom, temperature, at each node. The interpolating functions are linear and 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 its 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. Fig. 2 represents the shape of each element. It is assumed perfect contact between rebars and concrete, being the nodes of both elements shared.



a) SOLID70, thermal analysis of steel. b) LINK33, thermal analysis of rebars. c) SOLID70, thermal analysis of concrete.



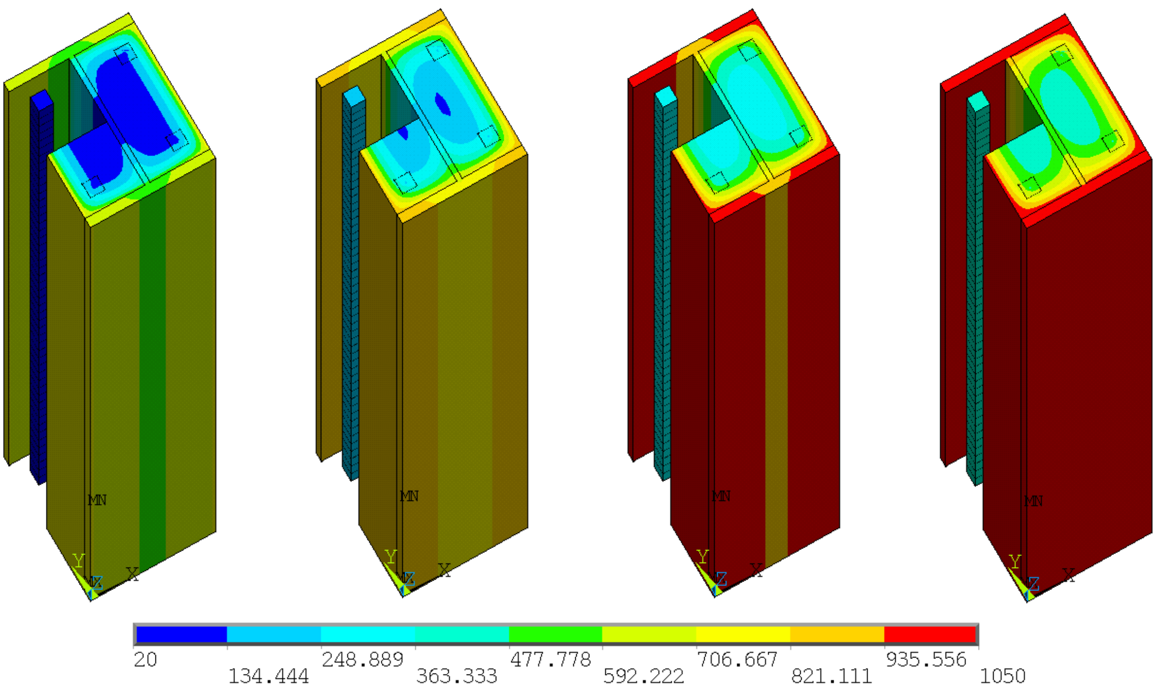
d) SOLID185, mechanical analysis of steel.

e) LINK180, mechanical analysis of rebars.

f) SOLID65, mechanical analysis of concrete.

Fig. 2: Finite elements used to build the three dimensional model of partially encased columns.

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 temperature field was determined for the total time of 7200 s. Fig. 3 shows an example of the partially encased column exposed to ISO834 fire, for 30, 60, 90 and 120 minutes.



a) Time=30 min. b) Time=60 min. c) Time=90 min. d) Time=120 min.

Fig. 3: Numerical results for column HEB 360 and for different fire ratings classes.

The temperature field was recorded for the corresponding resistance class and applied as body load to the mechanical model. The mesh was defined after a convergence test of the solution.

The critical load was determined by an eigen buckling analysis, using the modification of finite element types. SOLID185 and SOLID65 were selected to model the steel profile and the concrete, while LINK180 was selected to model the rebars. SOLID185 is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. This element has linear interpolating functions and can be used with different types of integration schemes. After a number of tests, reduced integration was selected. SOLID65 is also defined by eight nodes and the same degrees of freedom. This element was defined without internal rebars and with full integration (2x2x2). LINK180 was selected to model the rebars. This element has two nodes and a three degrees of freedom. The interpolating functions are linear and one integration point is used.

A static linear analysis is the basis for a general buckling analysis. The solution of Eq. 13 must be find primarily, where $\{F_{ref}\}$ is an arbitrary load on the structure, 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 used forces $\{F_{ref}\}$, which can be used to form the stress stiffness matrix $[K_{\sigma,ref}]$. Since the stress stiffness matrix is proportional to the load vector $\{F_{ref}\}$, an arbitrary stress stiffness matrix $[K_{\sigma}]$ and an arbitrary load vector $\{F\}$ may be defined by a constant λ as shown by Eqs 14-15.

$$[K]\{d\}=\{F_{ref}\} \tag{13}$$
$$[K_{\sigma}]=\lambda[K_{\sigma,ref}] \tag{14}$$
$$\{F\}=\lambda\{F_{ref}\} \tag{15}$$

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 Eq. 16, 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. 16-17 are still valid, where $\{\delta d\}$ represents the incremental buckling displacement vector.

$$[[K]+\lambda_{cri}[K_{\sigma,ref}]]\{d\}=\lambda_{cri}\{F_{ref}\} \tag{16}$$
$$[[K]+\lambda_{cri}[K_{\sigma,ref}]]\{\{d\}+\{\delta d\}\}=\lambda_{cri}\{F_{ref}\} \tag{17}$$

The difference between Eq. 16 and Eq. 17 produces an eigenvalue problem, represented by Eq. 18 where the smallest root defines the first buckling load, when bifurcation is expected.

$$[[K]+\lambda\ [K_{\sigma,ref}]]\{\delta d\}=\{0\} \tag{18}$$

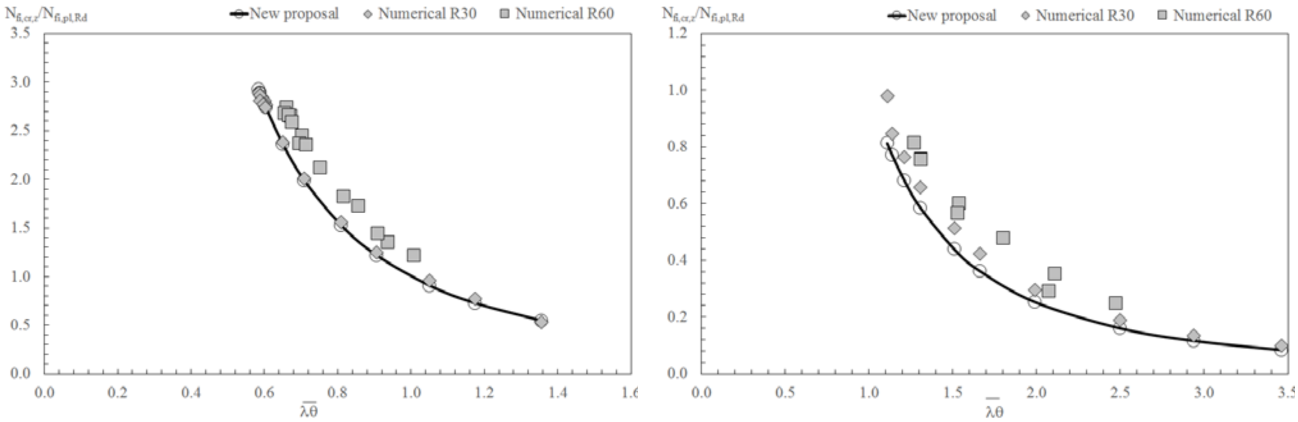
The trivial solution is not of interest which means that the solution for λ is define for an algebraic equation, imposing the determinant of the global matrix equal to zero. The calculated eigenvalue is always related to an eigenvector $\{\delta d\}$ called a buckling mode shape. This numerical solution 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.

Results of the critical load

The Euler buckling or elastic critical is compared by two different solution methods. The simplified solution method applies to Eq. 2. The non-dimensional slenderness ratio may be calculated using Eqs. 19-20, when the safety partial factors are equal to 1.0. The advanced solution method applies to the results of the buckling analysis, using the finite element method.

$$\overline{\lambda}_{\theta}=\sqrt{N_{fi,pl,Rd}/N_{fi,cr,z}} \tag{19}$$
$$N_{fi,pl,Rd}=N_{fi,pl,Rd,f}+N_{fi,pl,Rd,w}+N_{fi,pl,Rd,c}+N_{fi,pl,Rd,s} \tag{20}$$

Fig. 4 presents the comparison of the buckling load results, using the new proposal and the numerical solution, for 30 and 60 minutes of fire exposure.



a) Ratio between critical and plastic resistance for HEB. b) Ratio between critical and plastic resistance for IPE.

Fig. 4: Comparison of the critical load between the new proposal and the numerical results.

Conclusions

Two different solution methods were applied to define the elastic buckling load of partially encased columns in case of fire. The new proposal is based on the balanced summation method presented by EN 1994-1-2, but using safer formulas for the components to account for a reduction in geometry and un update of the material properties based on its average temperature. The numerical solution method is based on the elastic buckling analysis, considering the 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 formulae.
This work is ongoing to compare the prediction of the buckling resistance of partially encased columns and to validate the best fitting curve for axial buckling load.

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