CILASCI

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5º CONGRESSO IBERO-LATINO-AMERICANO EM SEGURANÇA CONTRA INCÊNDIOS

5th IBERIAN-LATIN-AMERICAN CONGRESS ON FIRE SAFETY

15-17 /07/ 2019 - Porto, Portugal

Atas dos Artigos
Proceedings (full papers)

ipb INSTITUTO POLITÉCNICO DE BRAGANÇA

U.PORTO FEUP FACULDADE DE ENGENHARIA UNIVERSIDADE DO PORTO

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PREFACE

The Iberian-Latin American Congress on Fire Safety (CILASCI) is held once every two years, with the aim of disseminating scientific and technical knowledge in the field of fire safety, integrating different players involved in this area of knowledge. The first edition of the Iberian-Latin American Congress on Fire Safety (CILASCI 1), was held in Natal (Brazil) between 10-12 March 2011. The second congress (CILASCI 2) was held in Coimbra (Portugal), between May 29 and June 1, 2013. The 3^{rd} and 4^{th} editions took place on the South American continent. The third congress (CILASCI 3) was held in Porto Alegre (Brazil) from November 3 to 6, 2015, while the fourth congress (CILASCI 4) was held in Recife (Brazil) from 9 to 11 October 2017. The CILASCI 5 will take place in the city of Porto (Portugal) from 15 to 17 July 2019, and presents 5 invited lectures and 78 manuscripts (full papers) from researchers around the world (Algeria, Australia, Belgium, Brazil, China, Czech Republic, France, Hong Kong, Italy, Mozambique, Portugal, Spain, United Kingdom and United States).

the 5^{th} Iberian-Latin-American congress on fire safety reflects the new developments achieved on active and passive fire protection, on evacuation and human behaviour under fire, on computational modelling of structures and materials under fire, on explosion and risk management, on architectural issues for fire safety in buildings, on fire dynamics, on the experimental analysis of materials and structures under fire, on fires in special buildings and spaces, on fire-fighting operations and equipments, and on the behaviour of structures and materials under fire.

The Fire Safety is reaching new developments as a result of new research, development and innovation around the world, based on the excellence level of the research, the support of new skilled professionals and due to the existence of advanced training programmes in fire science technology. This development will increase the safety level of people, buildings, and products, but also is going to produce an impact in the economy of each country, with a positive impact on society.

The organizing committee believe that this congress will address to our delegates a wide forum of discussion about the recent developments in Fire Safety, promoting the exchange of ideas and international cooperation.

The organizing Committee would like to thanks to all authors and delegates.

On the behalf of the Organizing Committe
Paulo A. G. Piloto

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PARTIALLY ENCASED COLUMNS EMBEDDED ON WALLS UNDER FIRE

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Keywords: Partially encased columns; Fire resistance; Columns embedded on wall; Numerical simulation.

1. INTRODUCTION

The steel columns have a great loadbearing capacity, however when exposed to fire conditions are heated in a relatively short period. The steel-concrete composite columns consist in an excellent alternative solution to bare steel column. Partially Encased Columns (PEC) have higher strength and stiffness when compared to steel bare profiles in fire conditions. The fire resistance of the PEC depends on the temperature evolution in each material and component, being this temperature field affected by the protection effect of the wall.

The annex G of Eurocode EN 1994-1-2 [1], presents the balanced summation method, allowing the calculation of the buckling resistance of PEC, when submitted to standard fire conditions from the 4 sides, but this annex does not take into consideration the embedded effect of this PEC in the wall.

The non-symmetric temperature filed over different types of materials and elements has been study for long time. In 1988, G.M.E. Cooke [2] presented the study, both experimental and theoretical, about the thermal bowing of building elements. The temperature field and gradient

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over the cross section depends on the material under analysis. Test data show that large thermal bowing deflections occur in unrestrained brick walls. Author also presented simple calculation models to deal with unrestrained thermal bowing effect on steel elements. The behaviour of composite elements is more complex due to the existence on non-linear thermal gradients in concrete. In 2007, Garlock and Quiel [3] presented the effect of the thermal gradients in steel columns with wide flanges, which is responsible for modifying the position of the neutral axis to the coldest side, introducing extra bending moments. This shift effect also depends on the amount of axial load. In 2010, António Correia et al [4] presented a new proposal for the calculation method to evaluate the temperature of H steel columns embedded in single-leaf brick walls. This proposal was based on numerical simulations and fire resistance tests, taking into to consideration two different relative positions between the steel profile and the brick wall. The proposal considered a temperature gradient, based on 3 isothermal zones or components (exposed, embedded and unexposed). In 2011, Dwaikat et al [5] tested several wide-flanged steel beam-columns with induced temperature gradients. These temperature gradients were induced by the effect of the furnace into a partial protected column, considering the inexistence of fire protection materials in certain regions, allowing to define two main directions for the highest gradient (parallel and perpendicular to the web). This procedure simulates, according to authors, a realistic three-sided heating scenario. The column specimens developed bending moments in regards to these gradients. Major gradients were defined in the direction parallel to the web for both fire events. All columns failed by full section yielding. The plastic resistance due to the combinations of axial load and moment was affected by the thermal gradients. The experiments and computer models showed good agreement with the predicted demands and capacity. In 2014, Agarwal et al. [6] made an experimental and numerical investigation to analyse the behaviour of columns with thermal gradients in the cross section. The columns subjected to uniform heating reached their failure temperatures faster than the columns subjected to non-uniform heating. The parametric analysis allowed for the development of design equations for wide flanged steel columns subjected to non-uniform heating. In 2014, Quiel et al. [7] investigated three different models to predict the axial plastic load bearing of steel columns with thermal gradients, using the code-based equations (American and European), using the fibre-beam element model and using the shell finite element model. Authors concluded that code-based equations are not satisfactory, since these equations do not consider temperature gradients. Both finite element models agree well with experimental results. The tests and models were very important to develop new simple formulas, which included de effect of moment reversal due to a shift in the section centre of stiffness, produced by the existence of temperature gradients. In 2014, António Correia et al. [8] presented a numerical study that considered non-uniform temperature distribution in the cross section of restrained steel H columns embedded on walls, validated by experiments, proposing new interaction formulas for axial force and bending moment. The contact of the steel columns with the walls is responsible for a big reduction in the temperature of the cross-section, leading to higher fire resistance when compared to ones observed in engulfed steel columns. According to the authors, these columns behave much more like beam-columns failing by bending, following the effect of the "thermal bowing", instead of failing by buckling. The Eurocode 3 formulation, considering a uniform temperature, within the cross-section, equal to the temperature of the exposed flange, leaded to a very conservative design. In 2016, Ojeda et al [9] studied the effect of thermal gradients over the cross-section of steel columns by means of finite element simulations. Authors investigated the eccentricity in the column, created by the temperature gradient and the reduction the flexural buckling resistance of the columns. Authors decided to simplify the calculation of the required parameters in order to handle a simple calculation method. A new design model was proposed, consisting of a set of simple equations which considers the eccentricity. The simplifications involve the calculation of the required material properties and geometric parameters at average, maximum or minimum temperatures in the section. Rocha et al., in 2018 [10] studied the fire behaviour of steel columns embedded on masonry walls with restrained thermal elongation. Authors found that the magnitude of the thermal gradients and their directions have a strong effect on the mechanical behaviour and stability of the PEC under fire. The thickness of the wall influenced the bending stiffness of the tested columns, affecting the restraining forces and displacements. The thermal gradient in the cross section is responsible for the thermal bowing effect, introducing additional bending moments and axial forces.

This study presents an new approximation method to define the axial buckling resistance of Partially Encased Columns, embedded on walls, under fire, taking into consideration the temperature effect over the plastic resistance to axial compression $N_{fi,pl,Rd}$ and over the effective flexural stiffness $(EI)_{fi,eff,z}$ around the minor axis.

2. PROFILES STUDIED

This work investigates partially encased columns embedded on clay walls under standard fire from only one side, see figure 1 and proposes and alternative method to the balanced summation model, including the contribution of more components, due to the existence of asymmetric temperature field, with respect to the plane defined by the web of the PEC.

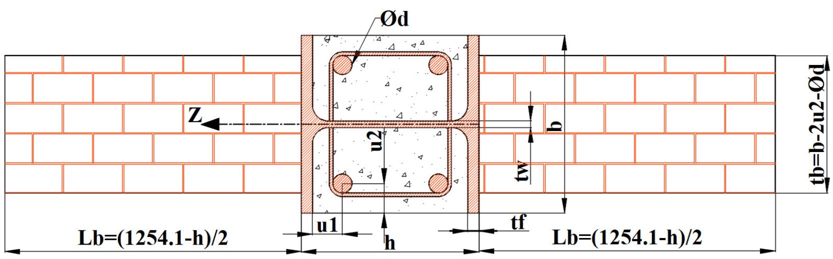


Figure 1: Partially encased column embedded on wall.

The thermal analysis of PEC studied in this work deals with 30 different columns, made by HEA, HD and UC cross section. The length and thickness of the wall changed, according to the values presented on Table 1. The grades of the materials were: S355 for the steel profiles, C20/25 for the encased concrete and B500 for rebars. A clay wall was considered on both sides of the partially encased section. The cross-section dimensions were selected to present a wide range of variation for the value of the section factor. The section factor was calculated by the exposed perimeter and the total area pf the cross section, following Eq. 1.

$$A_m / V = \left[(b - tb) + h \right] / b\ h \tag{1}$$

The balanced summation model, presented in annex G [1], was originally develop to determine the loadbearing capacity of PEC under fire, dividing the cross section in four components, based on the assumption of doubly symmetric thermal behaviour. In the following formulae, the four components of PEC are identified with, "f" for flanges, "w" for web, "c" for concrete and "s" for the reinforcing bars. The effect of each component for the calculation of the plastic resistance to axial compression can be calculated using Eq. 2 (normal summation) and the effective flexural stiffness using Eq.3 (balanced summation with reduction coefficients ϕ).

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

$$\left(EI\right)_{fi,eff,z} = \phi_{f,\theta}\left(EI\right)_{fi,f,z}\,\phi_{w,\theta}\left(EI\right)_{fi,w,z} + \phi_{c,\theta}\left(EI\right)_{fi,c,z} + \phi_{s,\theta}\left(EI\right)_{fi,s,z} \tag{3}$$

Table 1: Section proprieties for the partially encased sections.

Profiles	h [mm]	b [mm]	Tw [mm]	Tf [mm]	Am/V [m-¹]	Nº of bars	ϕ [mm]	As/ As + Ac	U [mm]	Rebar area [mm²]	Wall length (Lb) [mm]	Wall thickness (tb) [mm]
HEA 240	230	240	7.5	12	6.30	4	20	2.62%	50.0	314.2	512.05	122.28
HEA 280	270	280	8	13	5.19	4	25	2.96%	50.0	490.9	492.05	157.84
HEA 320	310	300	9	15.5	4.65	4	25	2.42%	50.0	490.9	472.05	177.84
HEA 360	350	300	10	17.5	4.56	4	32	3.52%	50.0	804.2	452.05	171.64
HEA 450	440	300	11.5	21	4.31	4	32	2.80%	50.0	804.2	407.05	171.64
HEA 500	490	300	12	23	4.21	4	32	2.52%	50.0	804.2	382.05	171.64
HEA 600	590	300	13	25	4.06	4	32	2.08%	54.8	804.2	332.05	171.64
HEA 700	690	300	14.5	27	3.99	4	40	2.77%	54.8	1256.6	282.05	164.55
HEA 800	790	300	15	28	3.90	4	40	2.40%	54.8	1256.6	232.05	164.55
HEA 1000	990	300	16.5	26	3.79	4	40	1.89%	54.8	1256.6	132.05	164.55
UC 254 x 254 x 73	254.1	254.6	8.6	14.2	5.82	4	25	3.54%	50.0	490.9	500.00	132.44
UC 254 x 254 x 167	289.1	265.2	19.2	31.7	5.36	4	25	3.54%	50.0	490.9	482.50	143.04
UC 305 x 305 x 158	327.1	311.2	15.8	25	4.41	4	25	2.40%	50.0	490.9	463.50	189.04
UC 305 x 305 x 313	374	325	30	48.3	4.08	4	25	2.40%	50.0	490.9	440.05	202.84
UC 356 x 368 x 202	374.6	374.7	16.5	27	3.58	4	32	2.80%	50.0	804.2	439.75	246.34
UC 356 x 406 x 340	406.4	403	26.6	42.9	3.27	4	32	2.67%	50.0	804.2	423.85	274.64
UC 356 x 406 x 592	465	421	45	72.3	3.03	4	32	2.67%	50.0	804.2	394.55	292.64
UC 356 x 406 x 744	498	432	55.6	88.9	2.91	4	32	2.67%	50.0	804.2	378.05	303.64
UC 356 x 406 x 1086	569	454	78	125	2.70	4	32	2.68%	50.0	804.2	342.55	325.64
HD 260 x 93,0	260	260	10	17.5	5.65	4	25	3.49%	50.0	490.9	497.05	137.84
HD 260 x 172	290	268	18	32.5	5.30	4	25	3.49%	50.0	490.9	482.05	145.84
HD 320 x 74,2	301	300	8	11	4.69	4	25	2.41%	50.0	490.9	476.55	177.84
HD 320 x 198	343	306	18	32	4.43	4	25	2.44%	50.0	490.9	455.55	183.84
HD 320 x 300	375	313	27	48	4.24	4	25	2.46%	50.0	490.9	439.55	190.84
HD 400 x 237	380	395	18.9	30.2	3.39	4	32	2.68%	50.0	804.2	437.05	266.64
HD 400 x 347	407	404	27.2	43.7	3.26	4	32	2.67%	50.0	804.2	423.55	275.64
HD 400 x 509	446	416	39.1	62.7	3.10	4	32	2.66%	50.0	804.2	404.05	287.64
HD 400 x 900	531	442	65.9	106	2.81	4	32	2.68%	50.0	804.2	361.55	313.64
HD 400 x 1299	600	476	100	140	2.55	4	32	2.67%	50.0	804.2	327.05	347.64

In the current version model, the average flange temperature is obtained through equations with empirical factors, depending on the section factor. The yield stress and elastic modulus are affected by temperature, using reduction factors. The web geometry reduction is based on empirical factors. The yield stress is reduced using an indirect parameter, leaving the elastic modulus not affected by temperature. The concrete temperature is calculated using a table based on the section factor, for each class of fire resistance. Part of the concrete geometry is neglected using the same distance in the both principal directions and the mechanical proprieties are affected by temperature. The reinforcing bars have the mechanical properties affected by temperature, using reduction factors for each size of concrete cover layer "u", obtained by the geometrical mean of "u_1" and "u_2". The material safety factors $\gamma_{M,fi,a}$, $\gamma_{M,fi,c}$ and $\gamma_{M,fi,s}$ for the structural steel, concrete and reinforcing steel, respectively, are assumed equal to 1,0.

3. NUMERICAL SIMULATION

The purpose of this work is to perform numerical simulations of composite steel-concrete columns in contact with clay walls under fire exposure from one side. The advanced calculation method was developed using the finite element method. The post processing of the temperature field of the PEC column allows to determine the temperature field of each component (exposed / unexposed flange, web, exposed / unexposed rebars, residual areas of the exposed and unexposed concrete). The standard fire ISO834 [11] is assumed to be applied around the one side of PEC.

$$\nabla(\lambda_{(T)} \cdot \nabla T) = \rho_{(T)} \cdot C_{p(T)} \cdot \partial T / \partial t \rightarrow (\Omega) \tag{4}$$

$$\lambda_{(T)} \cdot \nabla T \cdot \vec{n} = \alpha_c \left(T_g - T\right) + \phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma \cdot \left(T_g{}^4 - T^4\right) \rightarrow (\partial\Omega) \tag{5}$$

In these equations: $\lambda_{(T)}$ represents the thermal conductivity, $\rho_{(T)}$ defines the specific mass, $C_{p(T)}$ defines the specific heat, T represents the temperature of each material, T_g defines the gas temperature of the fire compartment, α_c defines the convective coefficient, ϕ defines the view factor, ε_f and ε_m defines the emissivity of fire and material respectively and σ represents the Stephan-Boltzmann constant.

The finite element model, considers the incremental and iterative solution, using 2D finite elements PLANE55. Nonlinear thermal properties are used for steel, concrete and masonry, according to the Eurocodes EN1993-1-2 [12], EN1994-1-2 [1], EN1992-1-2 [13] and EN1996-1-2 [14]. The arithmetic average nodal temperature was used to obtain the calculation of the average temperature of the components and the 500ºC isothermal criterion was used to define concrete layer to be neglected, $b_{c,fi,h}$, see Figure 2.

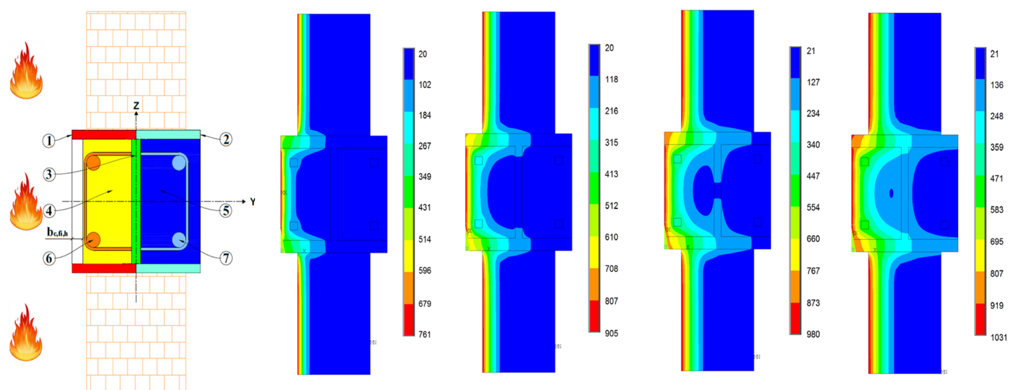


Figure 2: Temperature distribution in four times of resistance and results obtained.

4. BALANCED SUMMATION MODEL – NEW PROPOSAL

The new proposal to be used in the balanced summation model requires analytical formulas to take into consideration the effect of the fire in seven components. The flange, concrete and rebars are divided in two components, exposed to fire and not exposed to fire. The new equations are present in Eq. 6 and Eq. 7. The reduction coefficients ϕ were considered equal to current version of EN1994-1-2, with exception to the coefficients $\phi_{f,\theta}$ and $\phi_{s,\theta}$ applied to R120, that should be equal to 0.8.

$$N_{fi,pl,Rd} = N_{fi,pl,Rd,f,e} + N_{fi,pl,Rd,f,ne} + N_{fi,pl,Rd,w} + N_{fi,pl,Rd,c,e} + N_{fi,pl,Rd,c,ne} + N_{fi,pl,Rd,s,e} + N_{fi,pl,Rd,s,ne} \quad (6)$$

$$(EI)_{fi,eff,z} = \phi_{f,\theta}(EI)_{fi,f,e,z} + \phi_{f,\theta}(EI)_{fi,f,en,z} + \phi_{w,\theta}(EI)_{fi,w,z} + \phi_{c,\theta}(EI)_{fi,c,e,z} + \phi_{c,\theta}(EI)_{fi,c,ne,z} + \phi_{s,\theta}(EI)_{fi,s,e,z} + \phi_{s,\theta}(EI)_{fi,s,ne,z} \quad (7)$$

4.1 FLANGE COMPONENTS

The flange is divided into two components, half-flange exposed to fire and the half-flange not exposed to fire. New formulae are proposed for the calculation of the average temperature, based on some geometrical factors. Table 2 presents the improved equations for the average temperature of the components, with its respective parameters for each series of profiles.

Table 2: New formulae and parameters for average flanges temperature.

θfe,t = θ0,t + kfe,f (tf)+ kt,fe (Am/V)									
Standard Fire Resistance	HEA			HD			UC		
	kf,e [°C/mm]	kt ,fe [m°C]	θ0,t [°C]	kf,e [°C/mm]	kt ,fe [m°C]	θ0,t [°C]	kf,e [°C/mm]	kt ,fe [m°C]	θ0,t [°C]
R30	0.09	36	185	-0.42	45.95	140	-0.55	40.9	150
R60	1.48	45	295	-0.40	59.6	254	-0.53	54.5	262
R90	2.25	54	348	-0.20	69.95	307	-0.34	62.5	326
R120	3.15	58	389	0.01	74	344	-0.20	67.0	375
θf,t = θ0,t + kf,fne (tf)+ kt,fne (Am/V)									
Standard Fire Resistance	HEA			HD			UC		
	kf,ne [°C/mm]	kt ,fne [m°C]	θ0,t [°C]	kf,ne [°C/mm]	kt ,fne [m°C]	θ0,t [°C]	kf,ne [°C/mm]	kt ,fne [m°C]	θ0,t [°C]
R30	2.18	22	-66.4	0.11	23.62	-38	0.10	21.8	-31
R60	5.19	41	-139.0	0.30	46.8	-63	0.30	42.0	-45
R90	8.39	65	-245.0	0.72	71.4	-110	0.79	66.2	-96
R120	10.20	85	-305.0	0.92	93.8	-144	0.98	83.8	-110

The reduction of the mechanical properties is made by the average flange temperature, using the reduction factors presented in EN1994-1-2 [1]. The calculation of the plastic resistance to axial compression and effective flexural stiffness for the flange components, using the temperature effect on the material properties, are represented in Eq. 8 and Eq. 9.

$$N_{fi,pl,Rd,f} = 2\left[\frac{\left(b/2 \; t_f f_{ay,f,e,t}\right)}{\gamma_{M,fi,a}}\right]^1 + 2\left[\frac{\left(b/2 \; t_f f_{ay,f,ne,t}\right)}{\gamma_{M,fi,a}}\right]^2 \quad (8)$$

$$(EI)_{fi,f,z} = \left[2\left(\frac{\left(t_f\,(b/2)^3\right)}{12} + \frac{t_f b^3}{32}\right)E_{a,f,e,t}\right]^1 + \left[2\left(\frac{\left(t_f\,(b/2)^3\right)}{12} + \frac{t_f b^3}{32}\right)E_{a,f,ne,t}\right]^2 \quad (9)$$

4.2 WEB COMPONENT

The reduction of the web area is not considered in this proposal, instead a new improvement is proposed, based on the average temperature of the entire web region. The new plastic resistance to axial compression and the effective flexural stiffness in this component use the temperature effect on the material properties (affecting the yield stress and elastic modulus), see Table 3 for the new parameters.

Table 3: New formulae and parameters for the web average temperature.

θw,t = θ0,w + kw,w (1/tw)+ kt,w (Am/V)									
Standard	HEA			HD			UC		
Fire Resistance	$k_{w,w}$ [mm°C]	$k_{t,w}$ [m°C]	$\theta_{0,w}$ [°C]	$k_{w,w}$ [mm°C]	$k_{t,w}$ [m°C]	$\theta_{0,w}$ [°C]	$k_{w,w}$ [mm°C]	$k_{t,w}$ [m°C]	$\theta_{0,w}$ [°C]
R30	90.93	22.00	-50	-251.00	23.80	22.00	-234.62	23.05	20.00
R60	344.12	36.00	-75	-694.57	49.00	30.00	-630.00	50.00	35.00
R90	212.59	64.00	130.	1220.32	79.00	45.00	1142.11	77.49	40.00
R120	578.77	73.00	145	1585.77	102.50	45.00	1500.00	103.00	45.00

The plastic resistance to axial compression and effective flexural stiffness for the web, without any geometry reduction, is presented in Eq. 10.

$$\begin{aligned} N_{fi,pl,Rd,w} &= \left[e_w\left(h - 2t_f\right)f_{ay,w,t}\right] / \gamma_{M,fi,a} \\ (EI)_{fi,w,z} &= \left[E_{a,w,t}\left(h - 2t_f\right)e_w^{\ 3}\right] / 12 \end{aligned} \quad (10)$$

4.3 CONCRETE COMPONENTS

The concrete has two components, concrete exposed to fire and concrete not exposed to fire. The average temperature of concrete component requires a new formulae and new parameters according to Table 4. The 500 ºC isothermal criterion was used to determine de maximum temperature and the region of concrete to be neglected. The concrete exposed to fire has an horizontal "$b_{c,fi,h}$" size reduction. There is no reduction to the unexposed concrete for any fire rating. The horizontal reduction of the concrete exposed to fire is present in Table 5.

Table 4: New formulae and parameters for average concrete temperature.

θce,t = θ0,t + kce(1/tw)+ kt,ce (Am/V)									
Standard	HEA			HD			UC		
Fire Resistance	Kce [°C/mm]	Kt [m°C]	θ0,t [°C]	kce [°C/mm]	Kt [m°C]	θ0,t [°C]	Kce [°C/mm]	Kt [m°C]	θ0,t [°C]
R30	-131	30	24	-37	28.0	33	-172	26.5	50
R60	26	40	39	-155	38.9	70	-444	46.5	60
R90	86	50	35	-449	58.9	75	-785	68.9	50
R120	424	50	55	-764	75.0	85	-943	79.5	60
θcne,t = θ0,t + kf,cne (1/tw)+ kt,cne (Am/V)									
Standard	HEA			HD			UC		
Fire Resistance	kcne [°C/mm]	Kt [m°C]	θ0,t [°C]	kcne [°C/mm]	Kt [m°C]	θ0,t [°C]	Kcne [°C/mm]	Kt [m°C]	θ0,t [°C]
R30	-32	10.2	-11.00	-70	10.0	-1	-100	10.50	-3.27
R60	79	22.7	-48.00	-272	27.6	-29	-348	27.80	-29.00
R90	76	39.0	-84.75	-535	47.0	-49	-654	48.00	-51.50
R120	75	53.9	-118.00	-683	56.9	-35	-888	54.80	-17.00

Table 5: New formulae and parameters for horizontal reduction of concrete exposed to fire.

$b_{c,fi,h} = b_{0,ch} + k_{w,ch}(1/t_w) + k_{t,ch}(A_m/V)$ if $\theta_{c,t}$ = 500°C, then $b_{c,fi,h} = (b - t_w)/2$									
Standard	HEA			HD			UC		
Fire Resistance	b0,ch [mm]	kt,ch [m^2°C]	kw,ch [mm°C]	b0,ch [mm]	Ktch [m^2°C]	Kwch [mm°C]	b0,ch [mm]	Ktch [m^2°C]	Kwch [mm°C]
R30	11.68	0	0	11.68	0.000	0	11.68	0.000	0
R60	25.4	0.05	17	23.25	0.026	-4	23.00	0.026	4.52
R90	28	0.34	-12	31.00	0.180	-12	31.00	0.23	-21
R120	31	0.25	65	38.00	0.400	-26	38.00	0.400	-27

The Eq. 11 and Eq. 12 present the formulae for the calculation of the plastic resistance to axial compression and the calculation of the effective flexural stiffness, for the concrete exposed and the concrete not exposed, respectively.

$$N_{fi,pl,Rd,c} = \left[\frac{0,86\left\{ \left(\frac{(h-2t_f)(b/2 - t_w/2 - b_{ce,fi,h})}{12} \right) - A_s \right\} f_{c,\theta}}{\gamma_{M,fi,c}} \right]^4 + \left[\frac{0,86\left\{ \left(\frac{(h-2t_f)(b/2 - t_w/2)}{12} \right) - A_s \right\} f_{c,\theta}}{\gamma_{M,fi,c}} \right]^5 \quad (11)$$

$$(EI)_{fi,ce,z} = \left[E_{c,sec,\theta}\left(I_{c,e,z} - I_{s,e,z} \right) \right]^4 + \left[E_{c,sec,\theta}\left(I_{c,ne,z} - I_{s,ne,z} \right) \right]^5 \quad (12)$$

4.4 REBAR COMPONENTS

The average temperature of reinforcing rebars require a new formulae and new parameters. The thermal behaviour of the reinforcing bars depends on its geometric position "u" and the section factor. The rebars are divided in two components, rebars exposed to fire and rebars unexposed to fire. The influence of geometric position "u" is higher for the case of rebars exposed to fire. The new parameters are present in Table 6.

Table 6: New formulae and parameters for the average temperature of rebars.

θs,e,t = θ0,s,e, t+ kt,s,e (Am/V)+ku,s,e(u)						
Standard Fire Resistance	HEA			HD / UC		
	θ0s,e,t [°C]	kt,s,e [m°C]	ku,s,e [m°C]	θ0s,e,t [°C]	kt,s,e [m°C]	ku,s,e [m°C]
R30	310	0,4	-2,992	140	6	0
R60	640	5,7	-6,719	245	20	0
R90	765	11	-7,400	360	22	0
R120	840	16	-7,600	420	26	0

θs,ne,t = θ0,s,ne, t+ kt,s,ne (Am/V)+ku,s,ne(u)						
Standard Fire Resistance	HEA			HD / UC		
	θ0s,ne,t [°C]	kt,s,ne [m°C]	ku,s,ne [m°C]	θ0s,ne,t [°C]	kt,s,ne [m°C]	ku,s,ne [m°C]
R30	20	3,5	-0,158	10	4	0
R60	35	12,0	-0,61	-10	16	0
R90	45	17,5	-0,60	-9	25	0
R120	-30	30,0	0,50	6	33	0

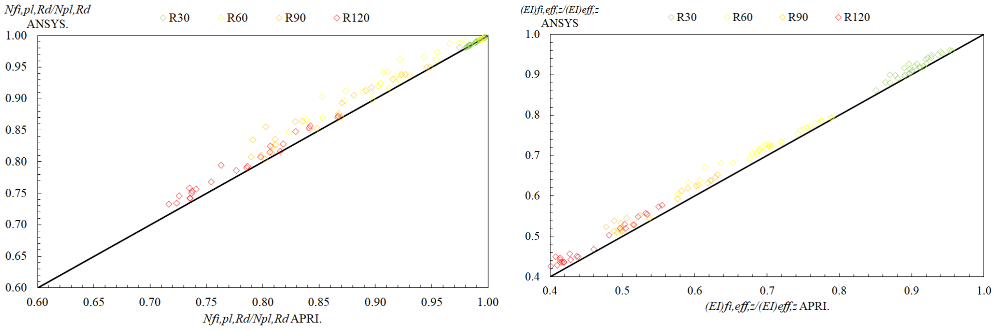
The Eq. 13 and Eq. 14 are used for the calculation of the plastic resistance to axial compression and the calculation of the effective flexural stiffness for these components. The reduction of the mechanical properties is introduced by the average temperature of rebars, using the reduction factors presented in EN1994-1-2 [1].

$$N_{fi,pl,Rd,s} = \left[\frac{A_{s,e} k_{sy,\theta} f_{s,y}}{\gamma_{M,fi,s}}\right]^6 + \left[\frac{A_{s,ne} k_{sy,\theta} f_{s,y}}{\gamma_{M,fi,s}}\right]^7 \tag{13}$$

$$(EI)_{fi,s,z} = \left[k_{s,e,E,q} E_s I_{s,e,z}\right]^6 + \left[k_{s,ne,E,\theta} E_s I_{s,ne,z}\right]^7 \tag{14}$$

5. RESULTS AND CONCLUSIONS

New simple formulas are presented, based on the average temperature of each component and based on the residual area of the concrete for different fire ratings (R30, R60, R90 and R120). The comparison of the results is presented in Figure 3, using the results of the finite element simulation and the results of the new formulae.



a) b)

Figure 3: Comparison results between the numerical simulation and improved formulae for the calculations of the resistance to axial compression (a) and effective flexural stiffness around the weak axis (b).

The simplified calculation method, proposed in annex G of EN1994-1-2, does not provide any formulae for the fire design of PEC embedded in walls. The new formulas, presented herein, consider the PEC column embedded in wall for the specific fire scenario (fire on one side). The wall protects the concrete of the fire effect, avoiding the reduction of the concrete in the direction of the web. The maximum reduction to the plastic resistance to axial compression, after 120 minutes of fire exposure is 72%, while the maximum reduction to the effective flexural stiffness is 40% for the same fire resistance.

The results obtained by numerical simulation to the plastic resistance to axial compression and to the effective flexural stiffness are bigger than the results obtained from the approximation, due to the fact that the approximation formulas are always providing higher average temperatures for each component. This higher temperature affects the mechanical properties (yielding and elastic modulus), leading to small values for the simple calculation method.

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