Temperature field of concrete-filled steel tubular columns in fire

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Abstract

The composite structures formed by the association of steel profiles with concrete have shown to be an advantageous alternative, improving the behavior of the steel structures concerning their load capacity, stiffness, and durability, under fire conditions. The concrete-filled tubular columns have become an attractive alternative for the civil construction area, offering constructive and esthetic advantages. The Eurocode 4 (EN 1994-1-2, 2002) and ABNT NBR 14323:2013 show simple methods for the design of the composite columns under fire situations. Nevertheless, these present limitations in their application. In addition, regarding the determination of the temperature field in the cross section, the Eurocode 4 (EN 1994-1-2, 2002) describes a specific procedure for columns with section profile covered with concrete, but it is neglectful about concrete-filled tubular columns. The goal of this paper is to present a simplified procedure to determine the temperature registered in concrete-filled steel columns and apply the procedure to columns with square section tubes. The temperature distribution was determined by simulations using the computer package ABAQUS (Dessault Systemes Simulia Corp. 2013). After the studies, it has been considered that the procedure described in this paper is an effective alternative for the determination of the field's temperature and for the application of the design of the concrete-filled columns under fire conditions.

Keywords: composite column, fire, temperature.

1. Introduction

The composite columns formed by steel tubes filled with concrete have shown to be an attractive alternative, for both esthetics and execution. Regarding its structural behavior, the tube with concrete provides an increasing resistance and stiffness to the structural element, minimizing the problems of instability, durability and fire resistance.

The buildings are vulnerable to the occurrence of a firestorm and, under the action of fire, the structural elements have their physical and mechanical properties changed with possible loss of load capacity and rigidity, which can lead to collapse, risking the physical integrity of the users.

The national normative codes, for the specific case of designing composite steel and concrete columns under fire conditions, describe tabular and simplified analytical methods that mention, briefly, advanced methods. Setting the temperature field in structural elements is critical for the design under a fire condition, and regarding composite columns with steel tubes filled with concrete, there are no practical procedures, neither in the national normative codes nor in the Eurocode.

Lie (1994) developed an analytical procedure to determine the temperature field in composite columns with circular section steel tubes filled with concrete. Lie and Irwin (1995) extended this procedure for composite columns with rectangular steel tube filled with concrete, however, such procedures have proved difficult for practical application.

Capilla (2012) researched the slender columns' resistance with circular and elliptical section steel filled with concrete under the action of fire, through numerical simulations, and described a simplified procedure proposed by Leskela (2009), to set equivalent temperatures in concrete and steel in those columns.

The process described in Capilla defines equivalent temperatures for the concrete core, the steel tube and the rebars, so that, the cross-section has the same resistance compared to the resistance obtained by the temperature field defined by advanced methods. Therefore, in this paper, considering the lack of practical procedures to set temperature fields in composite columns with steel tubes, we carried out an analysis of the procedure described in Capilla (2012), applying it to columns with square section steel tubes, filled with concrete and in fire situations.

The dimensions of the columns were, initially, based on the columns tested in Brazil, at the Universidade Estadual de Campinas, as shown in a work developed by Sant'Anna (2009).

List of the symbol

A_c : cross-sectional area of the concrete core

$A_{c,\theta,i}$: cross-section area of the layer "i" of the concrete core

E_c: elasticity modulus of concrete at the room temperature

$E_{c,\theta}$: elasticity modulus of concrete at the temperature θ

E_a: elasticity modulus of the structural steel at the room temperature

$E_{a,\theta}$: elasticity modulus of the structural steel at the temperature θ

E_s: elasticity modulus of reinforcing steel at the room temperature

$E_{s,\theta}$: elasticity modulus of reinforcing steel at the temperature θ

$E_{c,sec}$: secant modulus of elasticity of concrete

$E_{c,sec,\theta,i}$: secant modulus of elasticity of concrete at the temperature θ, applied to the "i" layer

$(EI)_{fi,c}$: flexural stiffness of concrete in the fire situation

f_c : compressive strength of concrete

$f_{c},_{\theta}$: compressive strength of concrete at temperature θ

$f_{c},_{\theta,i}$: compressive strength of concrete at temperature θ, applied to the "i" layer

f_{ya}: yield strength of structural steel

$f_{ya,\theta}$: yield strength of structural steel at temperature θ

f_{ys}: yield strength of reinforcing steel

$f_{ys,\theta}$: yield strength of reinforcing steel at temperature θ

I_c : second moment of area of the concrete

$I_{c,\theta,i}$: second moment of area of the concrete at temperature θ, applied to the "i" layer

$k_{c,\theta}$: reduction factor of the resistance of concrete at temperature θ

$k_{c,\theta,i}$: reduction factor of the concrete strength at temperature θ, applied to the "i" layer

$k_{Ec,\theta,i}$: reduction factor of the modulus of elasticity of concrete at temperature θ, applied to the "i" layer

$N_{fi,Rd}$: plastic resistance in axial compression of the concrete core in fire situation

ε_{cu} : corresponding deformation of the concrete to the peak stress

$\varepsilon_{cu,\theta}$: corresponding deformation of the concrete to the peak stress at temperature θ

$\varepsilon_{cu,\theta,i}$: corresponding deformation of the concrete to the peak stress at temperature θ, applied to the "i" layer

$\theta_{c,eq,num}$: equivalent temperature determined for the concrete from numerical models

$\theta_{a,eq,num}$: equivalent temperature determined for the steel tube from numerical models

2. Material and method

2.1 Equivalent temperature

To obtain the field temperature in composite columns with square sections, numerical simulations about heat transfer were carried out for analyses by using the models from the computer package ABAQUS (Dessault Systemes Simulia Corp. 2013).

Regarding the properties of the concrete and steel under fire conditions, those indicated in EN 1994-1-2 (Eurocode 4) were considered, and when it came to conductance, the equation of the upper limit for the concrete was considered.

To set an equivalent temperature for the concrete core, the core was divided into layers, and the area of each layer "i" was identified by $A_{c,\theta,i}$. This area is associated with a temperature θ_i, where θ_i is the temperature presented in each layer in the numerical models, taking the given points from the vertex $\theta_{i,a}$, whose values at some points are higher than the points indicated from the edge $\theta_{i,b}$, as shown in Figure 1.

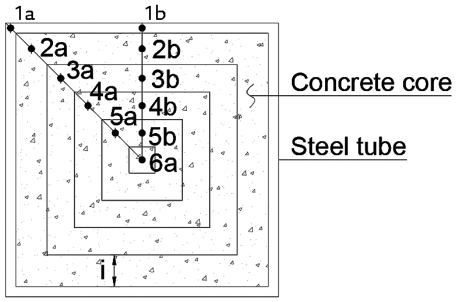


Figure 1
indication of temperature
monitoring points in the cross section.

Nevertheless the moment of inertia of each layer "i" is identified by $I_{c,\theta i}$ and the equivalent temperature is determined considering the plastic strength and the one direction flexural rigidity of the section.

Considering the equation of the plastic resistance in axial compression of the concrete core under fire conditions, Equation (1) was defined by the equivalent coefficient for reduction of section resistance:

$$N_{fi,Rd} = \sum_{i=1}^{n}\left(A_{c,\theta,i} \,.\, k_{c,\theta,i} .\, f_c\right) = f_c \sum_{i=1}^{n}(A_{c,\theta,i} . k_{c,\theta,i})$$

$$fc \sum_{i=1}^{n}\left(A_{c,\theta,i} . k_{c,\theta,i}\right) = k_{c,\theta}\left(\theta_{c,eq}\right). fc \,.\, Ac \rightarrow k_{c,\theta}\left(\theta_{c,eq1}\right) = \frac{\sum_{i=1}^{n}(A_{c,\theta,i} \,.\, k_{c,\theta,i})}{A_c} \tag{1}$$

Where the coefficient $k_{c},_{\theta}(\theta_{c,eq1})$ was defined by considering the subdivision of the concrete core in "n" layers with their respective average temperatures. From this coefficient, we can determine the equivalent temperature by interpolating the values shown in Table 1.

θ_c (°C)	$K_{c,\theta} = f_{c,\theta} / f_c$	$K_{Ec,\theta} = E_{c,\theta}/E_c$	$Ka,_\theta = fya,_\theta / fya$	$KEa,_\theta = Ea,_\theta/Ea$	$Ks,_\theta = fys,_\theta / fys$	$KEs,_\theta = Es,_\theta/Es$
20	1.00	1.000	1.000	1.000	1.000	1.000
100	1.00	0.625	1.000	1.000	1.000	1.000
200	0.95	0.432	1.000	0.900	1.000	0.870
300	0.85	0.304	1.000	0.800	1.000	0.720
400	0.75	0.188	1.000	0.700	0.940	0.560
500	0.60	0.100	0.780	0.600	0.670	0.400
600	0.45	0.045	0.470	0.310	0.400	0.240
700	0.30	0.030	0.230	0.130	0.120	0.080
800	0.15	0.015	0.110	0.090	0.110	0.060
900	0.08	0.008	0.060	0.0675	0.080	0.050
1000	0.04	0.004	0.040	0.0450	0.050	0.030
1100	0.01	0.001	0.020	0.0225	0.030	0.020
1200	0.00	0.000	0.000	0.000	0.000	0.000

Table 1
Reduction factors for the concrete with volumetrical normal mass and rolled steel and drawn steel.

The Table 1 shows values of the strength reduction coefficient, $K_{c,\theta}$, $K_{a,\theta}$, $K_{s,\theta}$ and the reduction factors of the longitudinal modulus of elasticity, $K_{Ec,\theta}$, $K_{Ea,\theta}$, $K_{Es,\theta}$, with respect to the concrete, steel tube and reinforcing bars, all factors related with the temperature.

All the coefficients shown in Table 1 were taken from Eurocode 4, except the $K_{Ec,\theta}$, that was calculated considering: $K_{Ec,\theta}= K_{c},_\theta .\varepsilon_{cu}/\varepsilon_{cu,\theta}$, where ε_{cu} is the specific deformation in room temperature, corresponding to 20 ºC and $\varepsilon_{cu,\theta}$, is the specific deformation at a given temperature, both listed in Eurocode 4 and corresponding to the compressive resistance of the concrete, as shown in Figure 2.

(a) Concrete temperature θ_c (°C)	$\varepsilon_{cu,\theta} \cdot 10^3$
20	2.50
100	4.00
200	5.50
300	7.00
400	10.00
500	15.00
600	25.00
700	25.00
800	25.00
900	25.00
1000	25.00
1100	25.00
1200	-

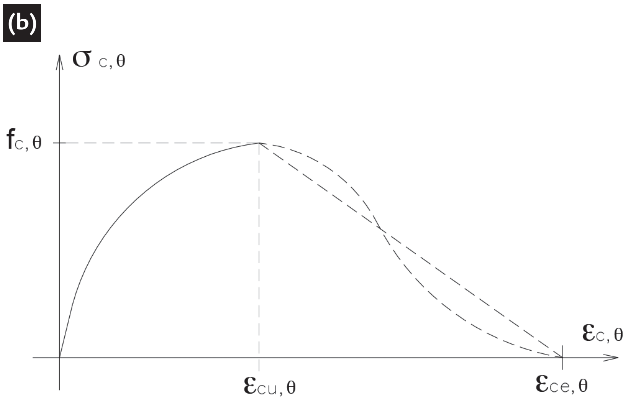


Figure 2
(a) ultimate specific deformation of the concrete in a temperature function;
(b) stress-strain function of concrete at high temperatures.

The equivalent ratio of reduced cross section rigidity can be determined as follows:

$$(\mathbf{EI})_{\mathbf{fi,c}} = \sum_{i=1}^{n}(\mathbf{E}_{\mathbf{c,sec,\theta,i}} \cdot \mathbf{I}_{\mathbf{c,\theta,i}}) = \sum_{i=1}^{n}(\frac{\mathbf{f}_{\mathbf{c,\theta,i}}}{\boldsymbol{\varepsilon}_{\mathbf{cu,\theta,i}}} \cdot \mathbf{I}_{\mathbf{c,\theta,i}}) = \frac{\mathbf{f}_{\mathbf{c}}}{\boldsymbol{\varepsilon}_{\mathbf{cu}}}\sum_{i=1}^{n}\left(\frac{\mathbf{k}_{\mathbf{c,\theta,i}}}{\frac{\boldsymbol{\varepsilon}_{\mathbf{cu,\theta,i}}}{\boldsymbol{\varepsilon}_{\mathbf{cu}}}} \cdot \mathbf{I}_{\mathbf{c,\theta,i}}\right) =$$

$$= \mathbf{E}_{\mathbf{c,sec}} \textstyle\sum_{i=1}^{n}(\mathbf{K}_{\mathbf{E,c,\theta,i}} \cdot \mathbf{I}_{\mathbf{c,\theta,i}}), \text{ whit } \mathbf{k}_{\mathbf{E,c,\theta,i}} = \frac{\mathbf{k}_{\mathbf{c,\theta,i}} \cdot \boldsymbol{\varepsilon}_{\mathbf{cu}}}{\boldsymbol{\varepsilon}_{\mathbf{cu,\theta,i}}} \quad (2)$$

$$\mathbf{EI}_{\mathbf{fi,c}} = \mathbf{E}_{\mathbf{c,sec}} \sum_{i=1}^{n}(\mathbf{k}_{\mathbf{Ec,\theta,i}} \cdot \mathbf{I}_{\mathbf{c,\theta,i}}) = \mathbf{k}_{\mathbf{Ec,\theta}}(\boldsymbol{\theta}_{\mathbf{c,eq2}}) \cdot \mathbf{E}_{\mathbf{c,sec}} \cdot \mathbf{I}_{\mathbf{c}}$$

$$\rightarrow k_{Ec,\theta}(\theta_{c,eq2}) = \frac{\sum_{i=1}^{n}(k_{Ec,\theta,i} \cdot I_{c,\theta,i})}{I_c}$$

The coefficient $k_{Ec,\theta}$ ($\theta_{c,eq2}$) is determined considering the "n" layers with their respective average temperatures.

From this coefficient, the equivalent temperature can be determined by interpolating the values shown in Table 1.

The temperature to be defined for the concrete core, conservatively, will be the higher of the two values: $\theta_{c,eq}$=max ($\theta_{c,eq1}$ e $\theta_{c,eq2}$).

The temperature for the steel tube is defined directly from the numerical model results by taking the average thickness of the tube and considering a small variation along the thickness. To set the temperature in the reinforced bars, a procedure similar to that of the steel tube was performed, but only considering the temperature regarding the position of the bars in the corner of the cross section and disregarding temperature changes depending on the tube thickness.

2.2 Numerical model

To obtain the temperature field through the ABAQUS software, we applied in the plane model:

a) Finite element formulation considering the basic equation of the energy balance together with Fourier law (DC2D4 – 4 nodes linear heat transfer quadrilateral);

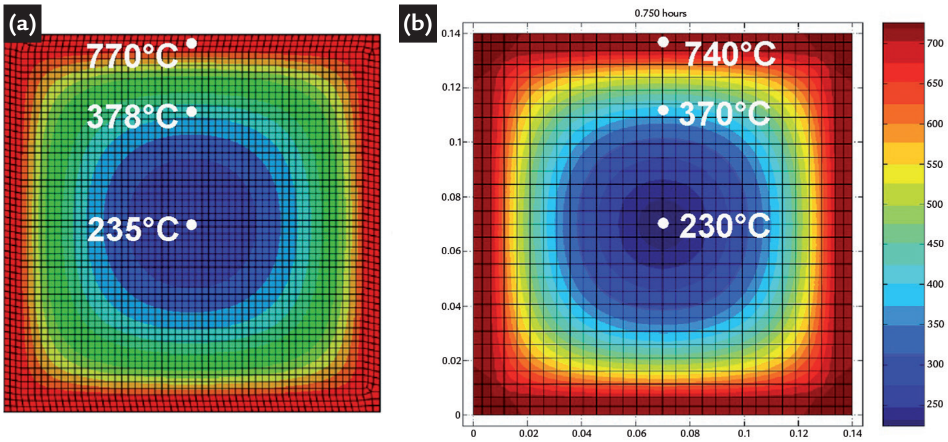
b) Standard fire curve (ISO 834) to heat the element, as thermal load applied at the four edges of the steel tube through radiation and convection mechanisms;

c) Radiation factor equal to 1, emissivity of the exposed face of the tube 0.7 and of the fire equal 1;

d) Convection coefficient for the exposure surface equal to 25 $Wm^2/^oC$ and Stefan-Boltzmann constant of $5.67x10^{-8}$ $Wm^{-2}K^{-4}$;

e) Density of steel and concrete 7850 kg/m^3 and 2300 kg/m^3, respectively, and the presumed actual moisture of 3%, considered through the specific heat equation.

The temperature fields determined in the numerical simulation were compared with the models listed in Sant'anna (2009) and as can be seen in the example of Figure 3, the values of the field temperatures were similar, having differences of less than 5% at the established points.

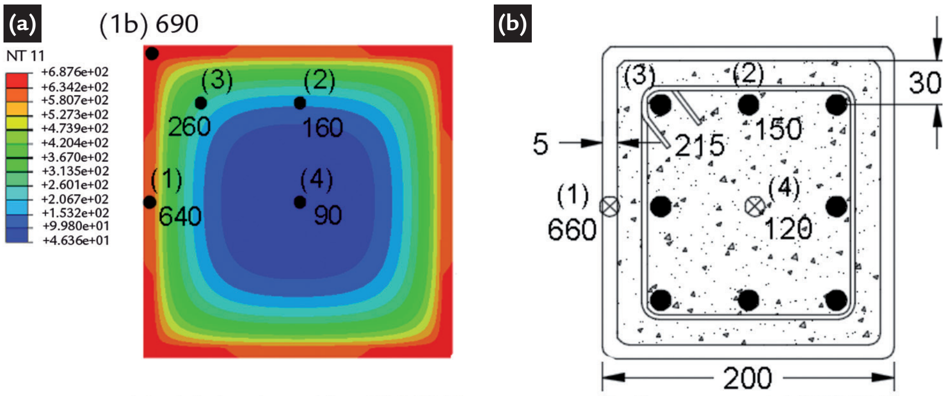


model developed

reference model

Figure 3
Comparisons of temperature profiles for a column L=140 mm, tube thickness = 6.3 mm and t =45 min.

Renaud (2004) shows a temperature spread study about square section columns under fire conditions, through experimental tests and simulations considering the material properties indicated in Eurocode 4. The results of the model (Figure 4b) were compared with the temperature profile set by ABAQUS (Figure 4a). The composite column consists of a steel tube filled with concrete with a length of L = 200 mm and thickness of 5 mm. The comparison was made for the fire exposure time of 30 minutes, without considering the resistance in heat conduction between the steel tube and the concrete, showing a good approximation regarding the temperatures at the points indicated.



Model developed by ABAQUS

Reference model (CTICM)

Figure 4
Temperature profiles obtained at 30 minutes of fire exposure.

The sections with the temperature field were sectored considering uniform divisions in layers with maximum thickness of 20 mm, and the thickness of the steel tube was not subdivided.

With the temperature field obtained by a numerical model, and taking into consideration the average temperatures in each sector, the coefficients and the equivalents temperatures were obtained $\theta_{c,eq}$ e $\theta_{a,eq}$ in order to determine the mechanical properties and load capacity of the columns.

2.3 Parametric study

Some of the material properties are presented in Eurocode 4 and others according to other standards with different values, for example, the coefficient of thermal expansion of the concrete, the concrete density and moisture content of the concrete. In order to verify the differences in the temperature of the steel tube and the concrete core, we prepared three-dimensional models for a tube length of 500 mm. Thermal analysis was performed for two composite columns with square section, as shown in Table 2 and Table 3. The models were developed considering the variation of concrete properties and the resistance on the thermal conduction between the steel tube and the concrete core. This resistance on the thermal conduction comes from a gap made from the differential expansion between the materials.

Table 2
Models for parametric study - properties of the concrete.

Variable properties of the concrete

Model	Thermal expansion coefficient of the concrete	Agraggate type	Thermal expansion of the steel	Concrete moisture	Concrete density
1	according Eurocode 4	siliceous	according Eurocode 4	3%	according Eurocode 4
2	according Eurocode 4	siliceous	according Eurocode 4	10%	according Eurocode 4
3	according Eurocode 4	siliceous	according Eurocode 4	3%	2300 kg/m^3
4	according Eurocode 4	siliceous	according Eurocode 4	3%	2400 kg/m^3

Table 3 presents the temperature in the middle of the concrete core and the temperature of the steel tube; temperatures that were taken from the monitoring points as shown in Figure 1: (point 6a) and (point 1b), respectively.

Table 3
Temperatures obtained in three-dimensional models.

Temperatures according to parametric analysis

Columns	Model	Temperature in the monitoring points (°C)		Exposure time in fire (min.)
		Point 1b	Point 6a	
PQ-200-5	1	905.0	138.0	80
	2	887.3	51.5	
	3	903.6	135.3	
	4	901.7	123.9	
PQ-140-5	1	854.0	277.8	50
	2	828.8	77.6	
	3	852.1	271.0	
	4	849.5	254.9	

According to Figure 5, differences in temperature in the middle of the concrete core and the steel tube can be seen taking into account the variations in the concrete properties. A composite column with a tube having a side dimension of 200 mm and thickness of 5 mm was named as PQ200-5. The composite column with a tube having a side dimension of 140 mm and thickness of 5 mm was named as PQ140-5.

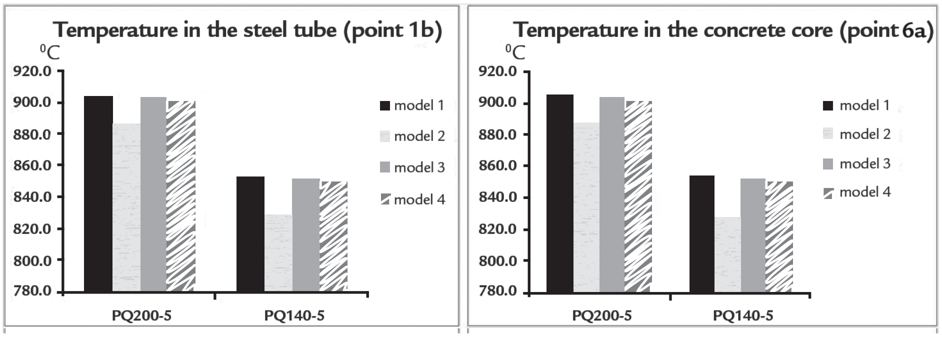


Figure 5
Results of three-dimensional models - Parametric study.

2.4 Study of thermal resistance between the steel tube and concrete core

The steel and the concrete have thermal distinct properties and they expand differently when submitted to high temperatures. The steel tube expands faster than the concrete core, which leads to a gap between the materials (Fig. 6).

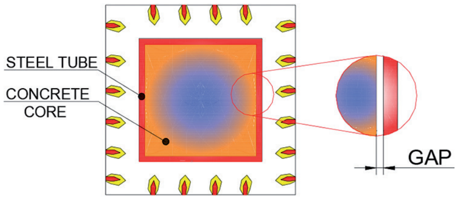


Figure 6
Gap between the steel tube and the concrete core in high temperature.

The gap affects the heating transfer by conduction between the steel tube and the concrete core, modifying the temperature field along the cross section. This effect can be considered by the software Abaqus, which stops the heat transfer by conduction when the contact between the steel tube and concrete core disappears, maintaining the heat transfer by convection and radiation.

What is possible to see in the available researches is the option of a perfect contact, without considering the gap and the thermal resistance between the tube and the concrete core or the adoption of a thermal dissipation coefficient or thermal resistance, which can be applied to the numerical models as a film at the interface between the two elements (tube and concrete). This coefficient must be adjusted by numerical or experimental tests.

To evaluate the resistance effect to the thermal conductivity in the interface between the steel tube and the concrete core, a tridimensional model was elaborated for the square section column named PQ200-5. The steel and the concrete properties were taken from Eurocode 4, and in this model the thermal contact conditions between the steel tube and the concrete core were altered. Three numerical analyses were elaborated considering different coefficients of thermal resistance, besides another one considering the perfect contact and a final one considering the thermal expansion of the materials.

The characteristics and results of the study of the thermal resistance can be seen in Table 4.

Temperatures obtained in the cross section (°C)

Column	Points	model with perfect contact	model with thermal resistance between the steel tube and the concrete core as resistence coefficient			model with thermal expansion	
			100 W/m²K	200 W/m²K	300 W/m²K		
PQ200-5	1	691	640	634	633	638	30 (min.)
	2	296	142	96	74	98	
	3	99	62	49	41	50	
	4	48	36	31	28	33	
	5	33	27	24	23	26	
	1	880	740	723	719	725	60 (min.)
	2	574	283	171	124	173	
	3	309	158	103	81	104	
	4	176	95	71	60	72	
	5	108	71	58	50	59	

Table 4
Temperatures obtained in cross section in the numerical analyses from the tridimensional models.

Figure 8 compares the values indicated in Table 4 regarding the temperature in the cross section for 30 minutes of fire exposure and Figure 9 for 60 minutes of fire exposure. The temperatures monitoring points in the tridimensional models indicated in the second column of Table 4 are also indicated in Figure 7.

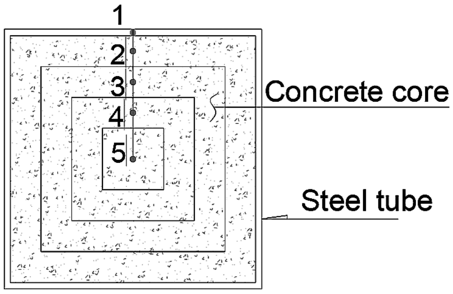


Figure 7
Temperatures monitoring points.

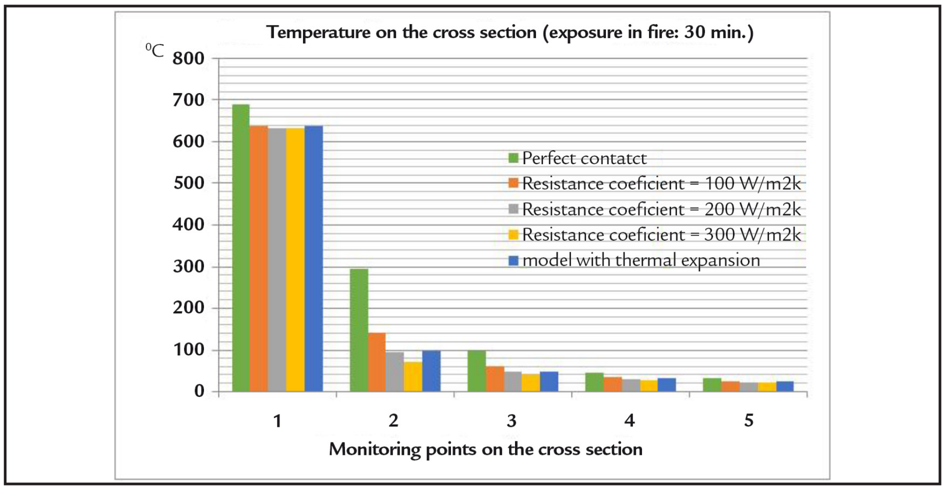


Figure 8
Temperatures in the cross section obtained for 30 min in fire exposure.

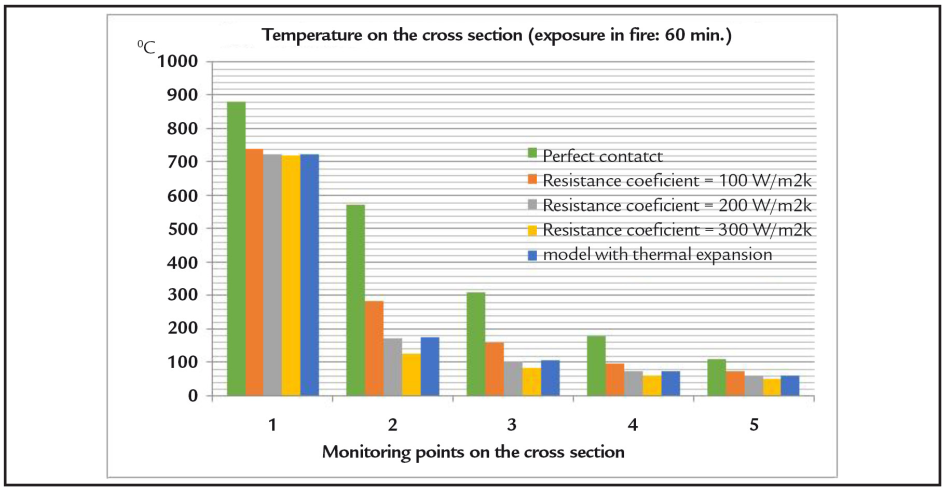


Figure 9
Temperatures in the cross section obtained for 60 min in fire exposure.

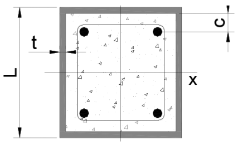
2.5 Simplified equation for the temperature field

A parametric study was prepared for the purpose of obtaining data to conduct a review and define the equations to set the equivalent temperature of the concrete core and steel tube and it was conducted considering the variations in the characteristics of the columns, as shown in Table 5 with a total of 48 samples.

Table 5
Values for the parametric study and cross-section of composite columns analyzed

PARAMETRIC STUDY

Features	Variables				
Tube - external dimension "L" (mm):	100	140	200	260	300
Tube thickness "t" (mm):	5.2	5.2	6.4	6.4	9.5
	6.4	6.4	9.5	9.4	12.7
	9.5	9.5			
Fire resistance "R" (min):	30 ; 60 ; 90 ; 120				



Based on the parametric study, the variables that significantly affect the temperature field were identified. By taking these variables into account, equations were defined for each fire resistance time. These equations were transformed into a single equation for the concrete core, the steel tube and the reinforced bars.

The plane models used to determine the simplified equations with the purpose of defining equivalents temperatures for the concrete, the steel tube and the reinforcing bars were elaborated considering what has been mentioned in item 2.2 and in the equations of 2.1.

3. Results

The equivalent temperatures obtained by numerical models, described at the concrete core and steel tube, are shown in table 6:

Table 6
Equivalents temperatures

			R30		R60		R90		R120	
Column	L/t	u/A (mm^{-1})	θc,eq,num (°C)	θa,eq,num (°C)	θc,eq,num (°C)	θa,eq,num (°C)	θc,eq,num (°C)	θa,eq,num (°C)	θc,eq,num (°C)	θa,eq,num (°C)
100-5.2	19.2	0.040	550	710	825	920	945	995	1010	1040
100-6.4	15.6	0.040	560	705	830	920	950	995	1010	1040
100-9.5	10.5	0.040	560	705	805	915	940	995	1005	1040
140-5.2	26.9	0.029	450	700	710	910	860	985	935	1030
140-6.4	21.9	0.029	445	690	720	905	855	985	935	1030
140-9.5	14.7	0.029	465	690	730	905	865	985	955	1030
200-6.4	31.3	0.020	395	670	575	890	710	970	820	1020
200-9.5	21.1	0.020	385	660	560	890	740	970	825	1020
260-6.4	40.6	0.015	325	670	525	880	580	960	685	1010
260-9.5	27.4	0.015	330	660	525	880	615	960	740	1015
300-9.5	31.6	0.013	360	630	515	870	595	955	705	1000
300-12.7	23.6	0.013	345	620	520	860	595	950	750	1000

The indicated code for the columns concerns the external dimension and the thickness of the tube.

It was observed that the pillars with smaller "L" heat up more quickly, and that the temperature variation in the concrete is less affected with the change of the tube thickness (chart a - Figure 10), but they are significantly affected when changing the massiveness factor, relationship between the perimeter, and the cross-sectional area (chart b - Figure 10).

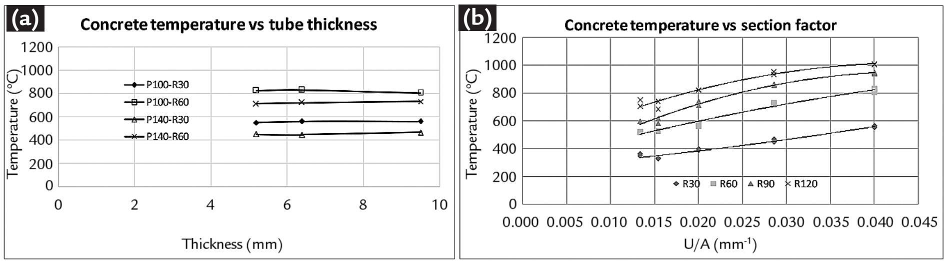


Figure 10
Chart (a) temperature x thickness tube and chart (b) temperature x section factor “u/A”.

As observed in the chart in Figure 10 (chart b), the polynomial quadratic function is the best fit to point cloud referring to temperatures on concrete samples, providing a strong correlation coefficient of 0.98.

However, for steel tubes, linear functions give fine results, as shown in the Figure 11.

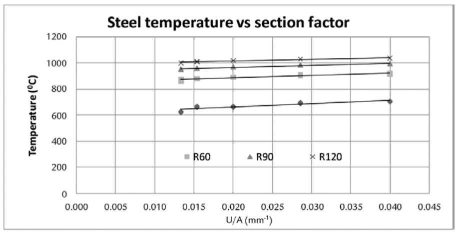


Figure 11
Chart of the temperature functions in the steel tube.

From the determined polynomial equations for each time of fire resistance, a single equation was defined to obtain the equivalent temperature in the core concrete as follows:

$$\theta c, eq = \left[(2.9892.\,R^3 - 605\,.\,R^2 + 29982\,.\,R - 374314)\,.\left(\frac{u}{A}\right)^2\right] + \left[(-0.1723\,.\,R^3 + 33.45\,.\,R^2 - 1495\,.\,R + 24399)\,.\left(\frac{u}{A}\right)\right] + 2.643.\,10^{-3}\,.\,R^3 - 0.545838\,.\,R^2 + 33.21\,.\,R - 317 \quad (3)$$

To determine the equivalent temperature in the steel tube, a simplified equation (4) was set by regression:

$$\theta a, eq = \left[(-2.2.10^{-3}.R^3 + 0.6393.R^2 - 67.R + 4023).\left(\frac{u}{A}\right)\right] + 7.17.10^{-4}.R^3 - 0.2127.R^2 + 22.54.R + 108 \qquad (4)$$

From temperatures in reinforced bars, a simplified equation (5) was defined by regression.

$$\theta s, eq = (0.015.R^3 - 3.89.R^2 + 346.R - 6003).\frac{u}{A} - 0.12.10^{-3}.R^3 - 0.014.R^2 + 9.3.R \qquad (5)$$

Where: "R" (min.) is the fire resistance time; "u/A" (mm-1) is the section factor of the cross section, "u" is the perimeter and "A" the area of the cross section.

4. Discussion

In the numerical plane model, used to produce the simplified equations, the resistance for heat conduction along the interface of the steel tube and the concrete core has been neglected, since it was considering the value of 200 W/m²K, recommended by Ding and Wang (2008). Significantly lower temperatures result, however the value indicated for the thermal resistance coefficient provided satisfactory results.

The Eurocode 4 shows limits for tube dimensions, but this will not be discussed in this work and to apply the proposed method to it, the dimensions and characteristics of the cross section and fire resistance time range specified in the table 5 should be followed; extrapolation is not valid.

Regarding the reinforced bars, it was observed that its inclusion in the model, practically does not change the temperature distribution in the cross section. Therefore, the temperatures in the rebars were considered the same as the concrete core in which the rebar's axis is encountered.

In composite columns with embedded reinforced bars, the equivalent temperature of the bars was defined taking into account a cover thickness of 3.0 cm. In higher covers, the elements will be exposed to minor temperatures and will be safer.

The equivalent temperature determined by equation (3) has a maximum error of 8.6% and average error of 1.3%. The simplified equation to determine the equivalent temperature in the steel tube (4) has a maximum error of 4% and an average error of 0.4% and the equation for the reinforced bars has a maximum error of 7% and an average error of 2.2% with reinforced bars at the corners.

The monitoring points of temperatures in numerical models plans, from the vertex of the cross section provided conservative values with the points set out from the middle of the edge, and there is less difference for longer periods of fire exposure. An intermediate value between the temperatures obtained in both points from the edge and from the vertex, could be adopted. However, in order to define conservative values, we chose to maintain the values for the temperature obtained by the path developed from the vertex (Figure 1).

Table 7 shows the equivalent temperatures for the concrete and the steel tube determined through the simplified equations shown in this paper, considering the temperatures taken from the vertex and from the edges.

Table 7
Equivalent temperatures for the concrete core and for the steel tube.

Equivalent temperatures for the concrete and the steel tube

Column	Exposure time in fire (min.)	u/a (mm^{-1})	Points i,a (from the vertex)		Points i,b (from the edge)		Diference	
			Concrete (°C)	Steel tube (°C)	Concrete (°C)	Steel tube (°C)	Concrete (°C)	Steel tube (°C)
PQ-200-6.4	30	0.020	395	670	285	620	27.8%	7.5%
PQ-140-5.2	30	0.029	450	700	390	655	13.3%	6.4%
PQ-200-6.4	60	0.020	575	890	570	870	0.9%	2.2%
PQ-140-5.2	60	0.029	710	910	645	890	9.2%	2.2%

To compare the answers obtained through the simplified equations, the tables shown in Renaud (2004) were used. These tables provide temperatures at determined points in the concrete core, in the steel tube and in the reinforcing bars and they were constructed according to the numerical models elaborated in the TASEF and SISMEF software. In these models, adapted were: a moisture of 4% of the concrete mass; thermal resistance between the steel tube and the concrete core of 0.01 m²K/W; reinforcing bars with ratio between 0 a 5% and emissivity of the exposed face of the tube of 0.6.

Table 8 displays the equivalent temperature values obtained by the simplified equations proposed in this paper and according to the tables in Renaud (2004).

Equivalent temperatures for the concrete and the steel tube (°C)

Columns	Exposure time in fire (minutes)	Simplified equations		Renaud		Diferences	
		Concrete	Steel tube	Concrete	Steel tube	Concrete	Steel tube
PQ-200-5	30	384	663	340	705	-12.94%	5.96%
PQ-140-5	30	452	684	400	715	-13.00%	4.34%
PQ-200-5	60	596	886	540	885	-10.37%	-0.11%
PQ-140-5	60	705	902	570	895	-23.68%	-0.78%

Table 8
Equivalent temperatures according to equations proposed and Renaud.

Temperatures in the reinforcing bars in the corners in columns of square sections

L (mm)	R (min)	u/a (mm^{-1})	Simplified equation (°C)	Renaud (°C)	Diferences
140	30	0.029	300	302	0.74%
200	30	0.020	289	288	-0.27%
300	30	0.013	280	287	2.36%
140	60	0.029	596	575	-3.61%
200	60	0.020	562	530	-5.95%
300	60	0.013	535	518	-3.27%

Table 9
Temperatures in the reinforcing bars according to equations proposed and Renaud.

The differences between equivalent temperatures determined by the simplified equations proposed in this article, and determined in Renaud (2004) are explained by the different parameters and properties adopted in the numerical models, and one should observe that the equations proposed considered, conservatively, the temperature set points in a path disposed from the vertex of the cross section.

5. Conclusion

The method and equations shown in this paper have provided satisfactory results for defining equivalent temperatures for the composite columns of a square section. The results, however, have shown to be conservative; first, because the thermal resistance between the steel tube and the concrete core was not taken into consideration, and second, because of the adopted parameters and the simplifications inherent to the procedure. This procedure can be extended to other types of usual cross sections. Nonetheless, the practical application of the method requires an analysis with more samples, taking into account the thermal properties of concrete and steel and considering adjustments based on the research conducted. The simplified equation to obtain the temperatures of the reinforced bars can have its results improved by using a larger number of numerical simulations to define an equation for the nonlinear regression, considering the other bars in addition to the ones in the corner of the cross section and with other cover rebars. The procedure shown in this paper is a complementary process to obtain temperature fields in composed columns and can be a starting point for discussions in future regulatory code review.

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