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ORIGINAL ARTICLE

Analysis of RC columns and shear walls in fire

Análise de pilares e pilares-parede de concreto armado ao incêndio

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Abstract: Recent disasters have shown that the failure of columns or shear walls exposed to fire is the main cause of building collapse. The cross-sectional thermal gradient (TG) of these structures justifies the degree of their heating and also justifies their mechanical damage, but the influence of the number of heated surfaces on their TG is not well known in the literature. A numerical FE analysis was proposed for RC columns and shear walls, assuming five fire cases in terms of the number of heated sides: only the (i) smallest and (ii) largest side, (iii) two, (iv) three and (v) four sides subjected to fire for 120 min. In order to validate the FE models, full-scale specimens were tested experimentally. Based on the number of sides hated, these structures can have an FRR of more than 120 min in relation to the values proposed by the standards.

Keywords: reinforced concrete, columns, shear walls, fire, structural design.

Resumo: Desastres recentes mostraram que a falha de pilares ou pilares-parede ao incêndio é a principal causa do colapso dos edifícios. Seu campo térmico justifica o dano mecânico, e o número de lados aquecidos não é plenamente entendido. Esta pesquisa propôs uma análise numérica de pilares e pilares-parede de concreto assumindo cinco casos de aquecimento da seção: apenas o (i) menor e (ii) maior lado, (iii) dois, (iv) três e (v) quatro lados submetidos ao incêndio por 120 min. Para validar os modelos numéricos, pilares em escala real foram testados em laboratório. Os resultados mostram que os pilares podem ter um TRF superior a 120 min em relação aos valores propostos pela norma.

Palavras-chave: concreto armado, pilares, pilares-parede, fogo, projeto estrutural.

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1. INTRODUCTION

Columns are important structural elements of reinforced concrete (RC) structures. In tall buildings, the magnitude of horizontal and vertical loads sometimes requires structural walls (or shear walls). According to ACI 318 [1], EN 1992-1.1 [2], and NBR 6118 [3], RC columns are referred to as shear or structural walls when the D/W ratio exceeds 3.0, 4.0, and 5.0, respectively. Due to their exceptional capacity to improve global structural stability, they are often used in high-rise buildings, just where fire safety requirements are more stringent [4]. In this context, structural engineers must consider the structural behavior in case of fire [5].

RC structures generally have good fire performance, as demonstrated by the Windsor Tower fire in Madrid, Spain, in 2005 or the Grenfell Tower fire in London, UK, in 201 [6]. However, recent fire disasters, such as the complete collapse of the Wilton Paes de Almeida building in Sao Paulo, Brazil, which collapsed after 90 min of fire in 2018,

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have changed this understanding. The global stability of high-rise buildings at high temperatures is influenced by the fire behavior of the columns [7], and their thermal behavior is essential to justify [8].

Research on structures in case of fire has received more attention in recent decades [9], but some authors emphasize that the thermal performance of shear walls in fire has not yet been fully investigated [7], [10], [11]. There is a lack of studies analyzing the temperature field in the cross-section of columns and shear walls, especially when they are exposed to fire on more than one side [11]–[13].

Shear walls in tall structures are frequently exposed to fire from only one side since they are typically used for shafts (stairs and elevators). This validates most of the research conducted in these conditions. On the other hand, basements, for example, might be utilized for parking and have columns and structural walls that are exposed to fire from multiple sides. The number of heated sides has an impact on the thermal gradient (thermal field) and fire behavior of the column and shear wall and needs to be investigated.

Lee and Lee [13] investigated the fire behavior of shear walls that are only exposed to fire from all sides. Experimental and FE studies were conducted to understand the effects of wall thickness, applied loading, concrete strength, and reinforcement ratio on fire resistance rate. The wall thickness and the mechanical loading were the most influential factors on the fire performance. Wall thickness had the greatest effect on the results relative to the other variables.

O'Meagher and Bennetts [14] developed numerical studies to analyze shear walls at high temperatures heated by one surface. They concluded that the behavior of these structures exposed to fire from a single surface can be significantly affected by the W/D ratio of their cross-section, and also the location of the steel reinforcement, the concrete cover thickness and boundary conditions. This is one of the first research papers on thermal modeling of RC columns.

The concrete cover thickness has received little attention from researchers [10], [15]. Reducing the concrete cover enhances the heating of the reinforcement and hence reduces the failure time of structures in fire [10]. In contrast, authors [6], [15] found that the FRR of shear walls decreases with increasing concrete cover thickness because the reinforcement is relocated near the barycenter of the cross-section, resulting in greater thermal curvature and lower flexural capacity during fire.

Ryu et al. [11] emphasize that the number of heated surfaces is critical because the heated region of the shear wall or columns induces eccentricity when only one surface is exposed to fire. According to Yang et al. [9], unilaterally heated columns tend to exhibit thermal curvature due to the difference in thermal expansion between heated and unheated surfaces, which leads to eccentricities in vertical loading and thus to second-order effects that impact global stability [16] [17].

Many authors [9], [11], [18], [19] have already investigated the thermomechanical behavior of shear walls in the event of fire, mainly with regard to thermal deflection and its correlation with the fire behavior of the structure. It is generally agreed that the thermal effects and the way in which the columns are heated influence their fire resistance. However, there is a gap regarding the influence of the number of heated surfaces and the resulting thermal gradient.

While EN 1992-1.2 [20] and NBR 15200 [21] specify fire design methods depending on the number of heated surfaces (assuming only two cases: one or two-sided heating), axial load, and wall thickness of the structure, ACI 216R-89 [22], AS 3600 [23], and NZS 3101 [24] consider only the thickness of the shear wall or columns as the main parameter for fire design. There is no consensus on which parameters are crucial for the fire behavior of shear walls and columns, which justifies the large number of studies that assume the thickness of these structures as the primary variable of research.

Recent research has investigated the mechanical damage to shear walls after fire [6], [25], [26]. Studies investigated the effects of boundary conditions on columns in fire based on FE or experimental investigations [19], [27]; their seismic performance at normal temperature [28]; the fire resistance after a seismic behavior [7] or after an explosion scenario [29]; the fire-induced progressive collapse [17] and the optimization of shear wall cross-section [4], the effects of shear walls on seismic performance of high-rise buildings [30], and others. The investigation of the thermal field in RC columns and shear walls is rarely the subject of research [31].

This paper shows the influence of the temperature field of columns and shear walls according to different heating scenarios: the fire was exposed only on (i) the smallest and (ii) the largest side, (iii) two, (iv) three, and (v) four sides. These structures were heated for 120 min according to the ISO 834 [32]. To validate the numerical models, a full-scale fire test was performed according to ASTM E 119 [33] and BS EN 1363-1 [34]. The width (W) and depth (D) of the cross-section, as well as the thickness of the concrete cover, are considered as variables, solved by Abaqus [35] software. A total of 80 FE models were solved based on the thermal parameters proposed in EN 1992-1.2 [20], NBR 15200 [36] and experimental data.

2 METHODS

The following notations are used in the following lines: C is the distance between the surface of the cross-section of the column/shear wall and the axis of the longitudinal reinforcement, CL is the column, D is the depth of the column or shear wall, D/W is the ratio of depth to width of a column or shear wall, FE is the finite element method, FEA is the finite element analysis, FRR is the fire resistance rate, RC is the reinforced concrete, SW is the shear wall, TRF is the fire resistance rate (in Portuguese: *Tempo de Resistência ao Fogo*) and W is the width of the column and/or shear wall. The term "thermal gradient" refers to the temperature difference, thermal field or temperature distribution along the cross-section of the column.

2.1 Cross-sections and nomenclatures

A total of 16 cross-sections were studied. The nomenclature of the columns or shear walls and their respective dimensions are shown in Table 1.

Table 1 – The column's characteristics and nomenclatures

Column name	Width "W" (thickness) (mm)	Depth "D" (mm)	D/W ratio	Standardized column classification*		
				EN 1992-1.1	ACI 318	NBR 6118
CL1-200x300	200	300	1.5	C	C	C
CL2-200x500	200	500	2.5	C	C	C
CL3-200x800	200	800	4.0	SW	SW	C
CL4-200x1000	200	1000	5.0	SW	SW	C
CL5-200x1200	200	1200	6.0	SW	SW	SW
CL6-200x1500	200	1500	7.5	SW	SW	SW
CL7-200x2000	200	2000	10.0	SW	SW	SW
CL8-200x3000	200	3000	15.0	SW	SW	SW
CL9-150x1000	150	1000	6.7	SW	SW	SW
CL10-250x1000	250	1000	4.0	SW	SW	C
CL11-300x1000	300	1000	3.3	C	SW	C
CL12-400x1000	400	1000	2.5	C	C	C
CL13-150x3000	150	3000	20.0	SW	SW	SW
CL14-250x3000	250	3000	12.0	SW	SW	SW
CL15-300x3000	300	3000	10.0	SW	SW	SW
CL16-400x3000	400	3000	7.5	SW	SW	SW

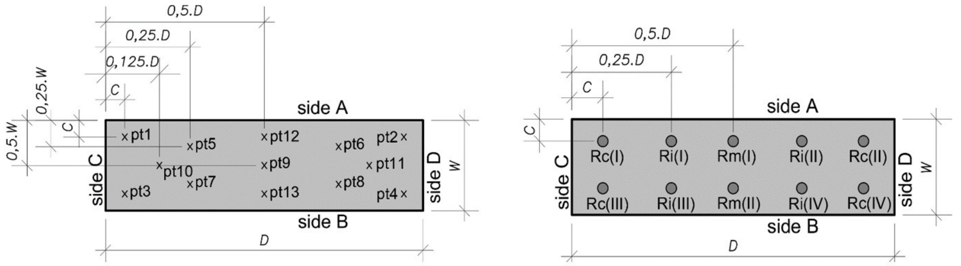
*Subtitle: C: column, SW: shear wall (also named as shear column)

The influence of the cross-sectional depth of the column on the thermal gradient was investigated for CL1 to CL8, each of which had a width of 20 cm. For a column with a width of 20 cm, the following cross-sectional depths were investigated: 30, 50, 80, 100, 120, 150, 200 and 300 cm. For CL4 and CL9 to CL12, the influence of the width of the column (15, 20, 25, 30 and 40 cm) was investigated at a depth of 100 cm. The same was carried out for CL8 and CL13 to 16, but at a depth of 300 cm.

The concrete axis distance "C" assumed was 30 mm. This distance refers to the axis of the longitudinal reinforcements to the column surface. Columns with 300 cm depth and 15, 20, 25, 30 and 40 cm width (CL8 and CL13 to CL16) were also studied with a "C" equal to 10, 20 and 40 cm. Some columns have the same D/W ratio but different D and W dimensions, such as CL2 and CL12 (D/W of 2.5), CL3 and CL10 (D/W of 4.0), CL6 and CL16 (D/W of 7.5), and CL7 and CL15 (D/W of 10.0).

2.2 Temperature control points

Figure 1 shows the temperature measurement in concrete and reinforcements. The average temperature of the concrete was determined using 13 measurement points, (pt1 to pt13 - Figure 1a). The average temperature of the rebars is defined using a total of ten rebars (Figure 1b) where R_c represents the corner rebars, R_m is the mean cross-section location and R_i is the intermediate position (between R_c and R_m).



(a) (b)

Figure 1 – Temperature control points on the (a) concrete and (b) reinforcements

2.3 Cases studied

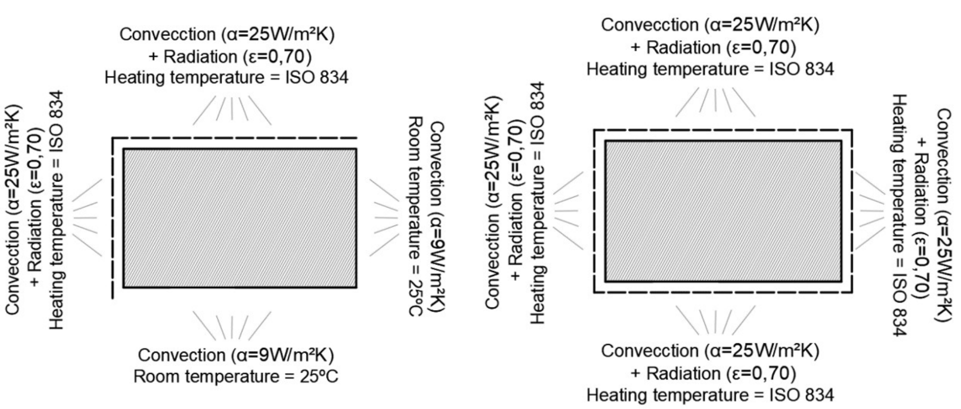
In the columns proposed in Table 1, a total of five heating cases were applied:

- Fire case I: one side fire-exposed (side A)
- Fire case II: one side fire-exposed (side C)
- Fire case III: two sides fire-exposed (sides A and C)
- Fire case IV: three sides fire-exposed (sides A, C and D)
- Fire case V: four sides fire-exposed (sides A, B, C and D)

2.4 Numerical models

The FE model is based on the thermal parameters for concrete and steel proposed in EN 1992-1.2 [2], EN 1993-1.2 [37] and is validated by the experimental data. The reinforcements were modeled with a wire of 2-node heat transfer link (DC1D2) and the concrete with a general 3D solid 8-node linear heat transfer brick (DC3D8) from the Abaqus library. A mesh analysis was performed and the mesh is rather refined, the size of the elements is 1.0 cm for DC1D2 and 1.0 x 1.0 x 1.0 cm for DC3D8. The values for thermal conductivity and specific weight of concrete were taken from EN 1992-1.2 [20]. The thermal conductivity and specific heat of the steel were taken according to EN 1993-1.2 [37], with a density of 7850 kg/m^3.

The ISO 834 [32] was applied in the surface by thermal convection (heat transfer α=25 W/m^2.K) and radiation (thermal emissivity Ɛ=0.70 [20]). On the unexposed one, an ambient temperature of 25ºC was applied by convection (α=9 W/m^2.K). Figure 2 shows the case of a column exposed to fire from two (a) and from four sides (b).



(a) two sides (b) four sides

Figure 2 – Number of fire-exposed surfaces

Since the cross-section was heated by the lateral surfaces, the length of the column has no influence on the thermal gradient. However, the numerical model of the column has a length of 1000 mm in all cases. In this sense, this method provides one result per unit length of the columns studied.

2.5 Numerical analysis criteria

The proposed numerical research focuses on the analysis of the critical temperature that causes mechanical damage. For steel, fib Bulletin No. 38 [38] proposes 500°C as the critical temperature, when the bending resistance moment in case of fire is of the order of 70% of the bending resistance moment at room temperature.

There are different suggestions about the critical temperature for concrete. Although [38] also assumes a critical temperature of 500°C from a mechanical point of view, authors such. [39] and [40] suggest temperatures in the range of 250 to 500°C because of the possibility of spalling, while [41] suggest 300°C. In this research, the spalling was neglected (assuming the same hypothesis as EN 1992-1.2 in their fire design tabulated procedures), and the focus was on mechanical properties, assuming a critical temperature of 500 °C for concrete and steel.

2.6 Correlation between standardized fire design methods and numerical results

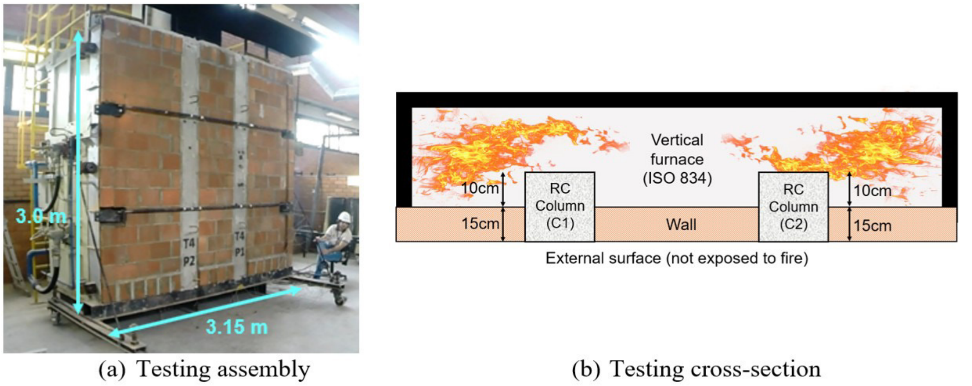
The numerical results were compared with the fire design methods proposed in EN 1992-1.2 [20], which form the basis of NBR 15200 [36]. The tabulated method was assumed, concerning Method A and B. Method A considers, in addition to the level of mechanical loading, the number of heated surfaces, minimum width of the column and also the distance between the longitudinal reinforcement and the surface of the cross-section. Method B is based on the same criterion, but, in addition, assumes the reinforcement rate without considering the number of heated surfaces. The method proposed for the structural walls uses the same criterion as Method A. Because this study was limited to thermal analysis, the EN 1992 [20] requirements resulting from the column's cross-section (i.e., minimum width and distance between longitudinal reinforcement and column surface) were considered.

2.7 Experimental validation

To validate the FE models, a real-scale test was carried out in a vertical furnace (Figure 3a). Experimental testing was performed with two specimens named as C1 and C2 according to ASTM E 119 [33] and BS EN 1363-1 [34] and following the ISO 834 [32].The C1 and C2 cross section was partially exposed to fire (Figure 3b).

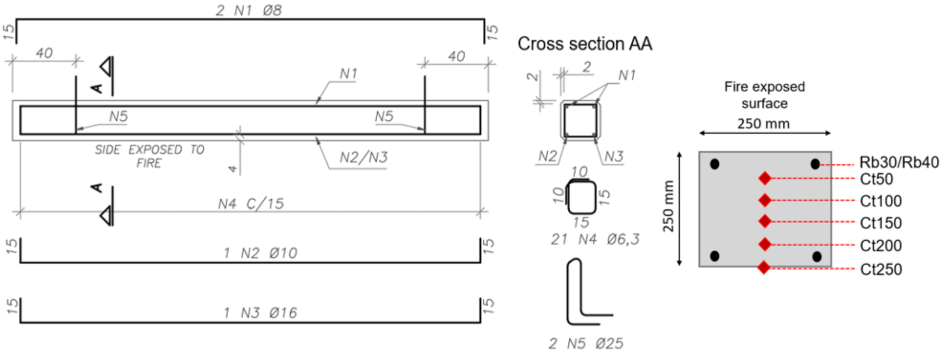
The structural drawing of the RC columns tested is shown in Figure 4a. The columns are 3000 in length (L) and have a 250 x 250 mm cross-section. The test included two real-scale columns (C1 and C2) with the same cross-section but concrete cover thickness “C” of 30 and 40 mm (Rb30 and Rb40). Each column was built assuming four longitudinal reinforcements, named as N1 to N3. N2 and N3 were placed on the surface exposed to fire, two N1 on the unexposed surface. The diameters of N1, N2 and N3 were 8, 10 and 16 mm, respectively. N4 was the transverse reinforcement, and N5 were the rebars used to move the columns. No mechanical load was applied.

To measure the temperature of the concrete layers, a total of 5 thermocouples were positioned at L/2 at a cross-sectional distance of 50, 100, 150, 200 and 250 mm from the heated surface. These thermocouples were designated as Ct50, Ct100, Ct150, Ct200, and Ct250 (Ct means “concrete thermocouples”), respectively (see Figure 4b). The concrete temperature was the average between C1 and C2 results, and in the case of reinforcement between N2 and N3.



(a) Testing assembly (b) Testing cross-section

Figure 3 – The vertical system (columns + walls) placed in the vertical furnace.



(a) Reinforcements (case of C=40 mm) (b) Thermocouples

Figure 4 – Structural drawings of the column experimentally tested

A Portland Brazilian cement classified as Type III according to ASTM C 150 [42] was used. The gravel was dacite with plagioclase and quartz-base with maximum dimensions of 9.5 and 19 mm. Natural quartz and industrialized dacite sand were also used in the concrete mixture. The compressive strength of the concrete specimens at 540 days was 24.3 MPa. These tests were performed according to ASTM C39 [43]. The rebars had a fy = 500 MPa.

To avoid heat transfer between the wall and the column, a rock wool (with very interesting thermal properties) was placed at the column-wall interface. The approach simplifies the FE model while simultaneously justifying the lack of a wall in the numerical model. For more details about the full-scale experimental test conducted by the corresponding author, see [44].

3 RESULTS AND DISCUSSION

3.1 Finite element (FE) model validation

The comparison between numerical model (num) and experimental (exp) specimens are shown in Figure 5 and Table 2.

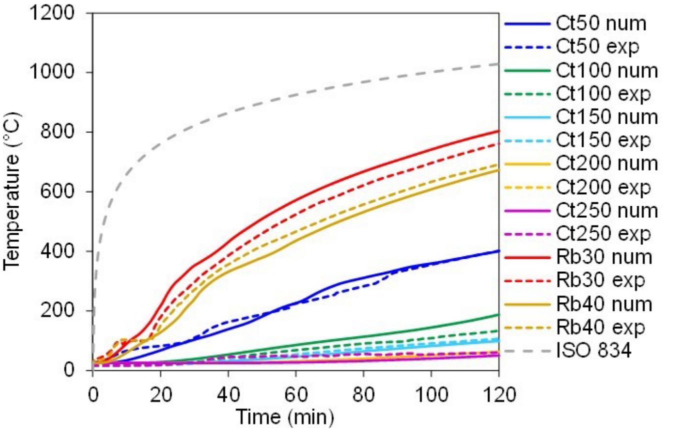


Figure 5 – Validation of the numerical models by experimental tests

The difference (Δ) between the experimental and computational models was less than 25.2 °C (see Rb40 at 90 min). Assuming that the concrete cracking and other variability in the experimental tests (such as thermal bowing, concrete spalling and others), it can be assumed that there is good convergence between the experimental and FE models, and the parameters assumed in the numerical procedures suggest the credibility of the results shown in the following sections.

Table 2 – Comparison between experimental (Exp) and numerical (Num) results

Case		ISO 834time-temperature (min)			
		30	60	90	120
		Temperature (°C)			
Ct 50	Exp	101.9	222.9	334.1	399.2
	Num	104.0	224.8	335.9	400.1
	Δ	2.1	1.9	1.8	0.9
Ct 100	Exp	35.7	79.3	115.1	169.7
	Num	38.7	84.9	127.3	187.6
	Δ	3.0	5.6	12.2	17.9
Ct 150	Exp	25.2	45.2	72.7	98.1
	Num	27.1	46.1	73.2	99.0
	Δ	1.9	0.9	0.5	0.9
Ct 200	Exp	25.0	31.7	45.9	64.8
	Num	25.1	30.6	44.6	63.1
	Δ	0.1	1.1	1.3	1.7
Ct 250	Exp	25.9	27.1	36.0	51.8
	Num	24.5	26.5	35.4	50.3
	Δ	1.4	0.6	0.6	1.5
Rb30	Exp	338.7	559.8	692.3	787.4
	Num	352.8	569.6	704.5	803.8
	Δ	14.1	9.8	12.2	16.4
Rb40	Exp	252.1	450.7	601.4	680.2
	Num	240.4	433.2	576.2	672.4
	Δ	11.7	17.5	25.2	7.8

3.2 Heating cases (FE model results)

3.2.1 Fire case I (one side fire exposed – side A)

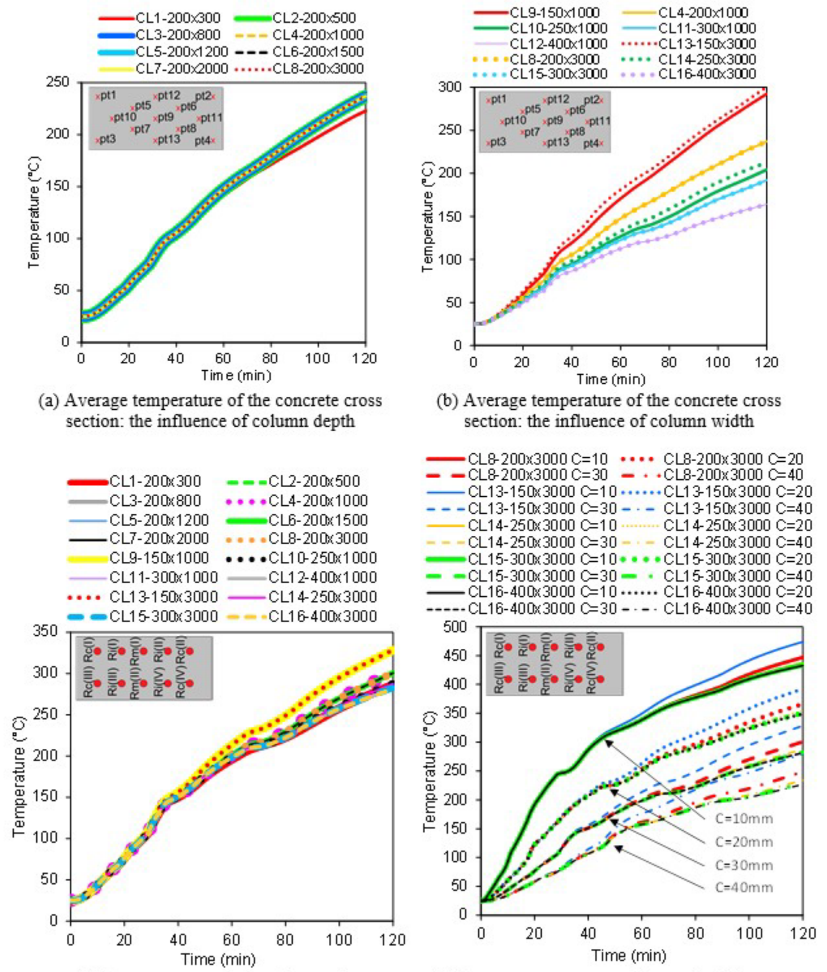
The temperature history in concrete and reinforcements are shown in Figure 6. The thermal gradient of some columns (CL2-200x500, CL3-200x800 and CL9-150x1000, with D/W equal to 2.5, 4.0 and 6.7, respectively) at 120 min are shown in Figure 7.

Figure 6a shows that if the “W” is maintained, the “D” has no influence on the results. However, if the "W" changes, the thermal gradient changes (Figure 6b). The progression of the heating– parallel to the “W” – is due to the identical heat diffusion in columns with identical width. This is can be seen from the isotherms of 200x500 and 150x1000 mm (see Figure 7a and b). The thermal gradient increases with decreasing "W". If “W” thickness increases, the average temperature decreases.

For W greater than 200 mm, the reinforcement of the side opposite the heated side (side B according to Figure 1) is no longer affected by the temperatures. Assuming W greater than 200 mm, the average temperature of the reinforcement is no longer affected by the column width. Columns with W=150 mm have a higher average temperature than the others.

Figure 6d shows that the average temperature of the reinforcement does not reach the critical temperature of 500 °C, regardless of the thickness of the concrete cover. The highest temperatures were measured in columns with C=10 mm. These rebars are more exposed to the high temperatures. In relation to C=10 mm, increasing the C thickness to 20, 30 and 40 mm reduced the temperature of the steel reinforcement by 85ºC, 151°C and 206°C respectively.

Regarding the EN 1992-1.2 tabulated procedure applied to columns exposed to fire on one side, Method A proposed by the standard highlights that columns with C=10 mm and 20 mm cannot be used as consequence of the fire safety requirements. The minimum C thickness standardized is 25 mm. However, the FE numerical tests have shown that the average temperature of the rebars with C=10 and 20 mm does not exceed 500 °C throughout the test and the shear wall supported 120 min of ISO 834, contradicting the standard. EN 1992-1.2 proposes a minimum “W” equal to 155 mm for this fire-case. However, in terms of the average temperature of the concrete and rebars, this study has shown that a thickness of 150 mm supports the standardized heating according to ISO 834 for at least 120 min.



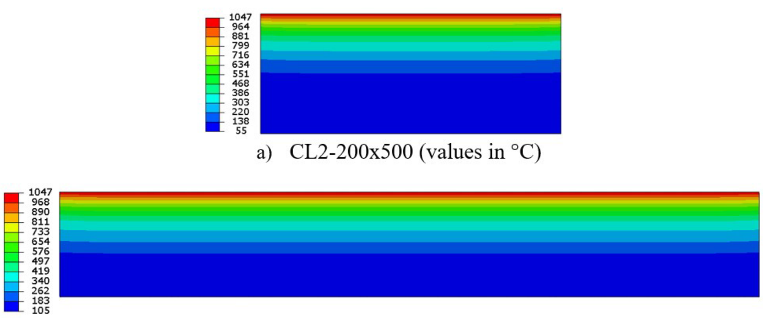
(a) Average temperature of the concrete cross section: the influence of column depth

(b) Average temperature of the concrete cross section: the influence of column width

(c) Average temperature of the steel reinforcement

(d) Average temperature of the steel reinforcement: for axis distance C of 10, 20, 30 and 40 mm (columns 3000 mm of depth only)

Figure 6 – Temperature for columns with one side fire exposed (fire case I-a)



a) CL2-200x500 (values in °C)

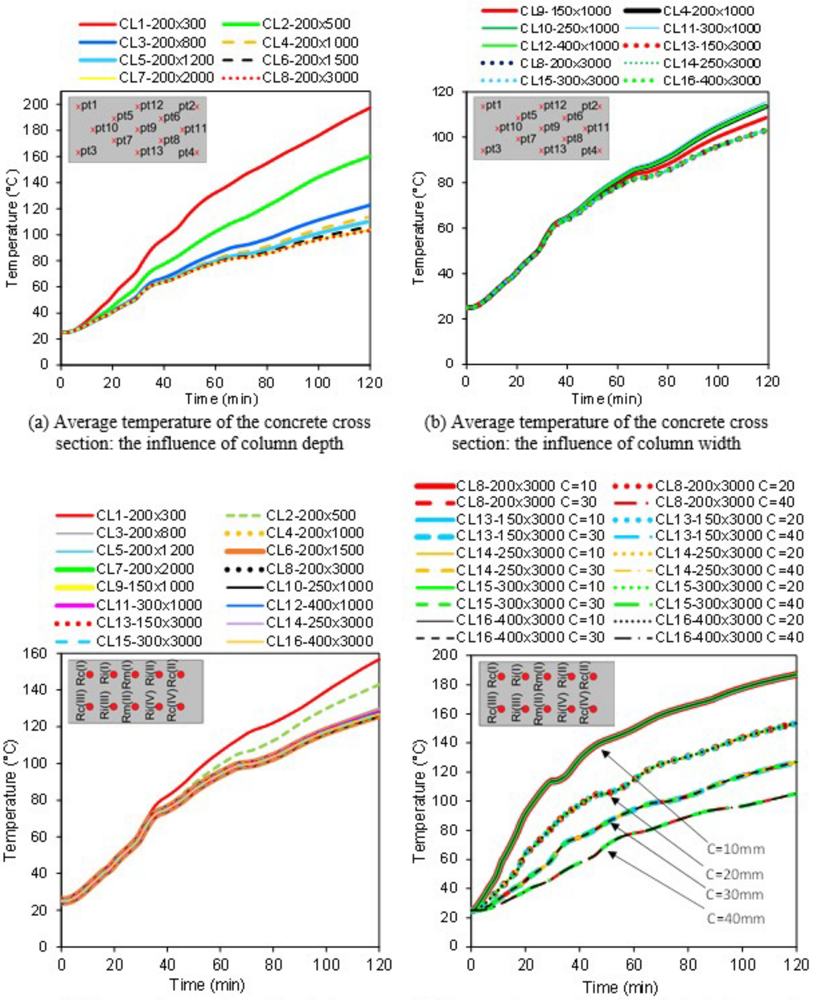
b) CL9-150x1000 (values in °C)

Figure 7 – Thermal gradient of column cross-section (fire case I) at 120 min

3.2.2 Fire case II (one side fire exposed – side C)

The temperature history in concrete and steel reinforcements over ISO 834 fire curve are shown in Figure 8. The thermal gradient is similar to that proposed in Figure 7, but heating the small side.

If the column is heated from the shortest side, the influence of "D" becomes clearer. Figure 8a shows that increasing "D" reduces the concrete average temperature up to a depth of 800 mm (D/W ratio of 3.0). After 1000 mm depth, it no longer has any influence on the results. The thermal gradient only exceeded 110°C after 120 min. When D/W < 3.0, this temperature can double and reaches 200°C after 120 min. Figure 8b shows that the "W" has no influence on the thermal results.



(a) Average temperature of the concrete cross section: the influence of column depth

(b) Average temperature of the concrete cross section: the influence of column width

(c) Average temperature of the steel reinforcement

(d) Average temperature of the steel reinforcement: for axis distance C of 10, 20, 30 and 40 mm (columns 3000 mm of depth only)

Figure 8 – Temperature for columns with one side fire exposed (fire case I-c)

Figure 8c shows similar conclusions for the rebars. The average temperature is not affected by the “D” up to 800 mm. In all columns, the average temperatures of the rebars do not exceed 500°C. Therefore, according to Figure 8c, columns with C=30 mm heated from one side are not severely damaged. The thermal gradient justifies the results, as a large part of the cross-section is at room temperature (approx. 25°C). The temperature of the rebars is influenced by the “C”. The temperatures were up to 300°C lower compared to the previous fire case. The rebars are less affected when the column is heated on its shortest side.

If the columns or shear walls are exposed to fire with only one surface, the thicker one, the tabulated values proposed by the standard are more conservative than in fire case I. All tested columns and shear walls (with a minimum width of 150 mm and C=10 mm) achieved a minimum FRR of 120 min. Heating the columns from the narrowest side is the most favorable situation, as the mechanical damage is minimal due to the lower thermal gradient (average temperature) in the cross sections.

3.2.3 Fire case III (two sides fire exposed – side A and C)

This section shows the results of heating the RC column from two sides (side A and C). The temperature history in concrete and steel reinforcements over ISO 834 curve are shown in Figure 9. The thermal gradient of some columns (such as CL2-200x500, CL3-200x800 and CL9-150x1000, with D/W ratio of 2.5, 4.0 and 6.7, respectively) at 120 min are shown in Figure 10.

Figure 9a shows that the thermal gradient in the cross-section is hardly influenced by “D”, as in fire cases I and II. Columns with a “D” less than 800 mm are more affected. When the longer side of the column is heated, its thermal gradient is not affected by the depth, and when the shorter side is heated, the average temperature is higher in columns with a depth of less than 800 mm.

Since the heating of the larger side predominates in relation to the smaller one, the thermal gradient in this fire case tends to be the same in all columns (Figure 9a). The results of the heating of the higher side are superimposed on the results of the lower side and increase the average thermal gradient in the cross-section. According to Figure 9a, D>800 mm have a slightly higher thermal gradient, which is due to the predominant heating on the shorter side in these cases (Figure 6a and Figure 8a).

The “W” influences the results more than “D” (Figure 9b). The thermal gradient is determined by the heating at depth (Figure 6a, Figure 8a and Figure 10). Figure 9c shows that the temperatures in the rebars are similar to those of the column heated only on the long side. Assuming C=30 mm, the temperatures in the reinforcements do not reach the critical 500 °C (Figure 9c).

Despite the heating of the larger side of the column being predominant in its thermal gradient, the heating of the smaller side was sufficient for the rebars to reach the critical 500°C. This was observed for a C=10 mm after 100 min of ISO 834 (Figure 9d). The most affected rebars were those of the column corners, especially Rc(I), which were heated from both sides (iA and C) (Figure 10). However, C=10 mm is unusual (are theorical) due to the environmental durability requirements of RC structures.

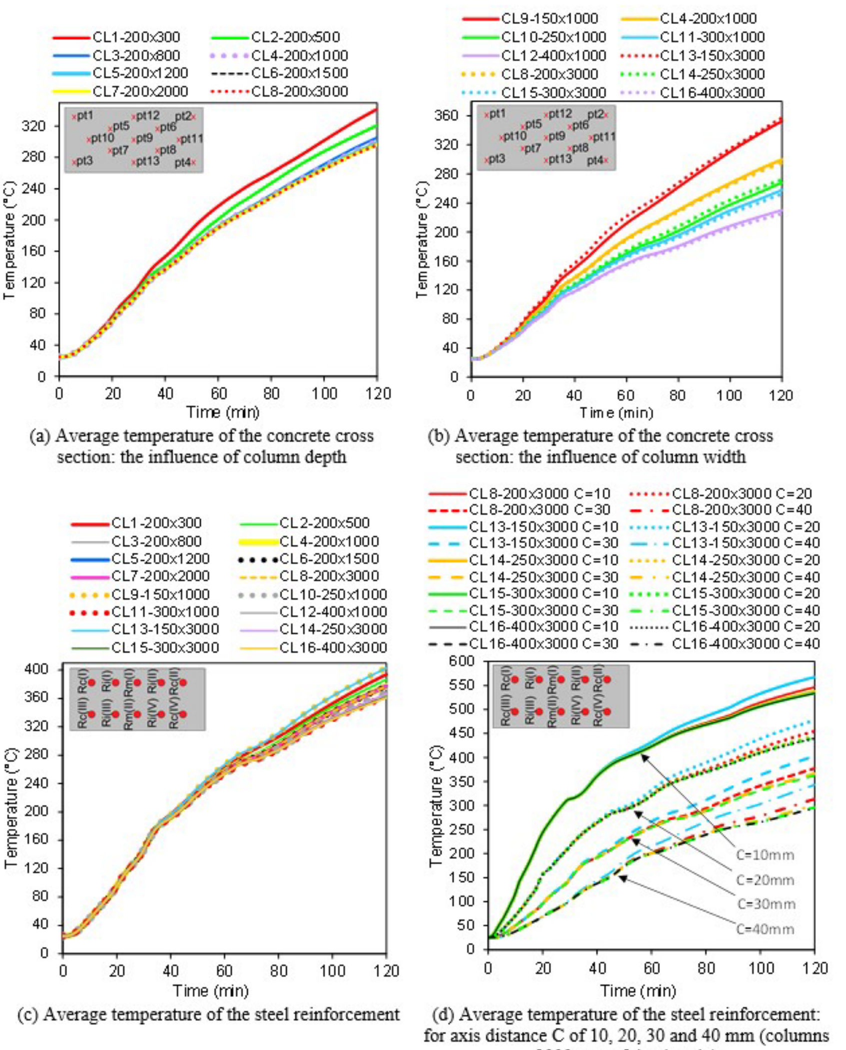


Figure 9 – Temperature for columns with two sides fire exposed (fire case III)

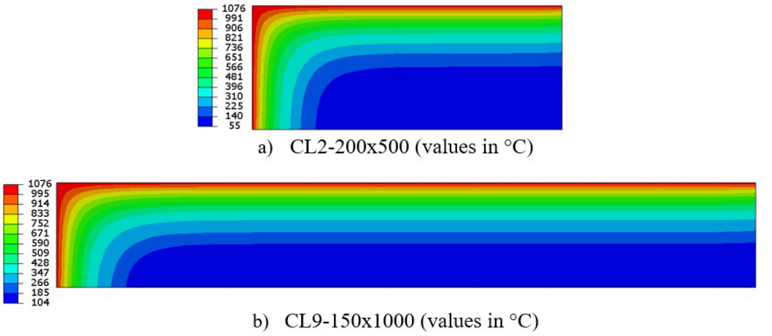


Figure 10 – Thermal gradient of column cross-section (fire case III) at 120 min

For the columns with W=200 mm and C of 10, 20, 30, and 40 mm, a maximum FRR= 0, 0, 90, and 90 min, respectively, was obtained according to Method A of EN 1992-1.2 (columns exposed to fire on more than one surface). According to the standard procedure, C=10 and 20 mm are unsafe and cannot be used. In the case of 30 and 40 mm with W=200 mm, an FRR=90 min is suggested in EN 1992-1.2 procedures. It's a contradiction with the FE results obtained in this research. Regarding W=200 mm and assuming the critical temperature of the reinforcements, Figure 9d shows that higher FRR are possible than those suggested by the standard. Even using a C=10 mm, it is possible to achieve an FRR of 90 min.

Method A of EN 1992-1.2 also not permits the use of columns or shear walls in fire with W=150 mm. In the FE numerical results, this width can reached a FRR up to 120 min in some cases, as shown in Figure 9d. In Method B proposed by EN 1992-1.2, columns and shear walls with a width of 150 mm can achieve an FRR of 30 and 60 min when C=25 and 30 mm, respectively, but with a very low mechanical load (maximum 15% of the total capacity). Considering only the critical temperature, this study shows that an FRR of more than 120 min can be achieved when W=150 mm.

3.2.4 Fire case IV (three sides fire exposed – side A, C and D)

The temperature history in concrete and steel reinforcements over ISO 834 fire curve are shown in Figure 11. The thermal gradient of some columns (such as CL2-200x500, CL3-200x800 and CL9-150x1000, with D/W ratio equal to 2.5, 4.0 and 6.7,) at 120 min are shown in Figure 12.

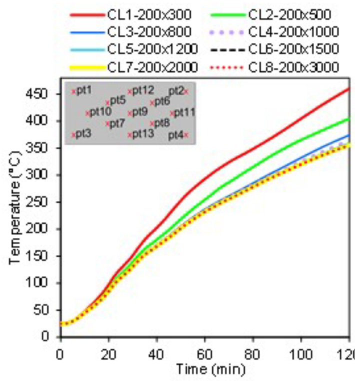
Figure 11a shows that the thermal gradient of columns with a depth of up to 800 mm has a higher average temperature than the other cases studied. However, after 800 mm width, the temperature of the other cross sections is usually the same. The depth of the column does not affect its thermal gradient. The highest temperatures at the depths of 300, 500, and 800 mm show that heating is critical at the shortest side below 1000 mm depth. The heating of the smaller side exerts less influence than heating of the larger side. The heating of the two small sides superimposes the heating of the larger side, increasing the thermal gradient in relation to another cases (Fire Case I to III). Figure 11b confirms that column width has a greater influence than depth.

According to Figure 11d, the "C" showed the greatest influence on the temperature. For a C=10 and 20 mm, an FRR of 60 and 90 min, respectively, was achieved. The increase in the temperature of the reinforcements is affected by the larger number of rebars that are heated. This can be seen by comparing the thermal fields of Figure 12 with those of Figure 7 and Figure 10.

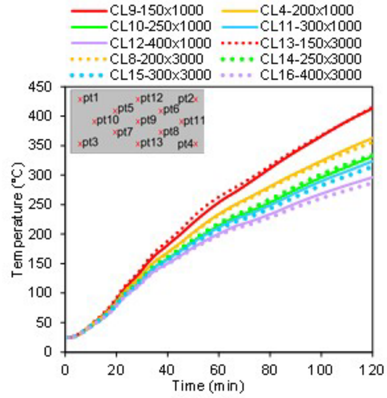
Comparing the results with the tabulated method proposed by EN 1992-1.2, it was found that the concrete did not exceed 500°C. For the average temperature of the rebars, it was found that, assuming C=10 mm, the critical temperature of the steel is reached at about 60 min. However, the application of C=10 mm is theorical.

EN 1992-1.2 suggests that the minimum width of shear walls or columns that can be used in tall buildings is 200 mm (Method A). A maximum FRR of 60 and 90 min is achieved for a C=25 and 31 mm, respectively. This study has shown that the use of columns or shear walls with W=200 mm can result in a higher FRR in relation to EN 1992-1.2. Assuming W=200 mm with C=20 mm, it is possible to achieve a FRR of 120 min, i.e., about twice the time proposed in the EN 1992-1.2.

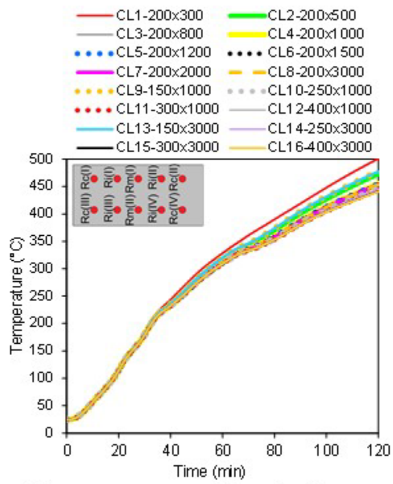
In the Method B of EN 1992-1.2, the minimum width of these structures, regardless of the assumed degree of reinforcement, is 150 mm and the C≥ 25 mm. This study has shown the opposite in relation to EN 1992-1.2. W=150 mm and C=20 mm can be used for a FRR=120 min. The standard focuses on the C thickness and the width of these structures. The standard clearly gives the lesser importance to the number of heated surfaces. This research has shown that the condition that most affects the FRR of these structures is the C and the number of heated surfaces, showing that the design philosophy of the standard needs to be corrected.



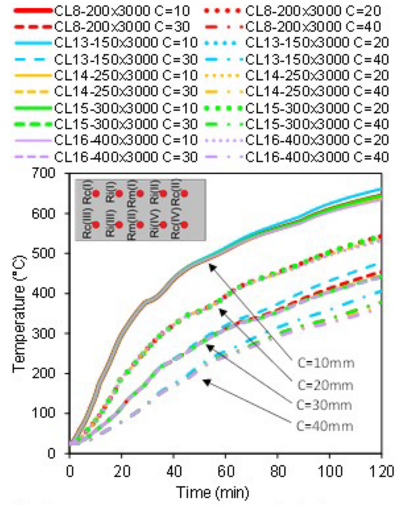
(a) Average temperature of the concrete cross section: the influence of column depth



(b) Average temperature of the concrete cross section: the influence of column width

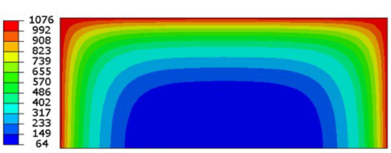


(c) Average temperature of the steel reinforcement

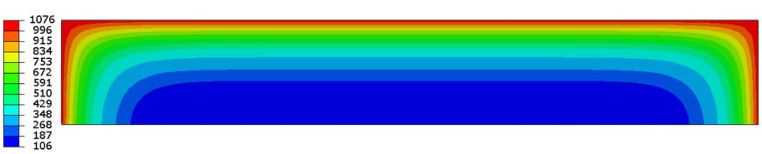


(d) Average temperature of the steel reinforcement: for axis distance C of 10, 20, 30 and 40 mm (columns 3000 mm of depth only)

Figure 11 – Temperature for columns with three sides fire exposed (fire case IV)



a) CL2-200x500 (values in °C)



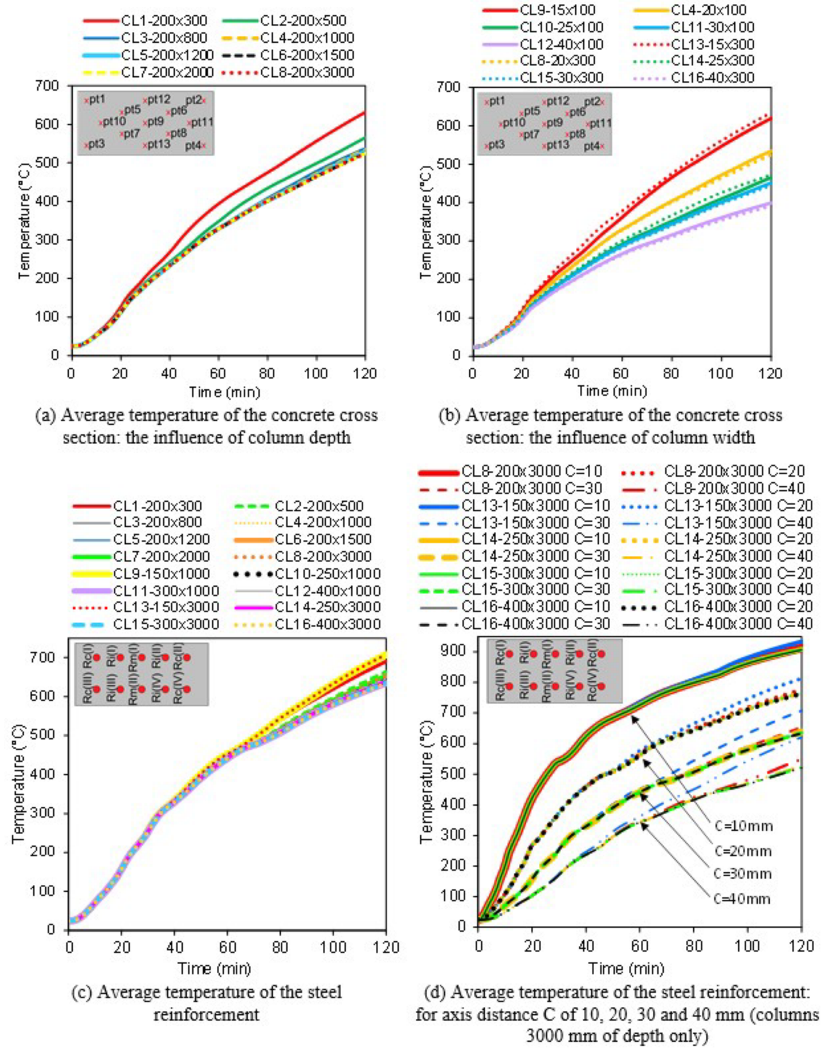
b) CL9-150x1000 (values in °C)

Figure 12 – Thermal field of column cross-section (fire case IV) at 120 min

3.2.5 Fire case V (four sides fire exposed – side A, B, C and D)

The evolution of the temperatures in the concrete and in the reinforcement during the ISO 834 heating are shown in Figure 13. The thermal gradient of some columns (such as CL2-200x500, CL3-200x800 and CL9-150x1000, with D/W ratio equal to 2.5, 4.0 and 6.7, respectively) at 120 min are shown in Figure 14. This is the most critical heating case.

Figure 13a shows that for conventional columns (i.e., up to 800 mm depth), depth affects the cross-section thermal field. After that, this variable no longer affects the results. This is a similar conclusion to the previous cases.



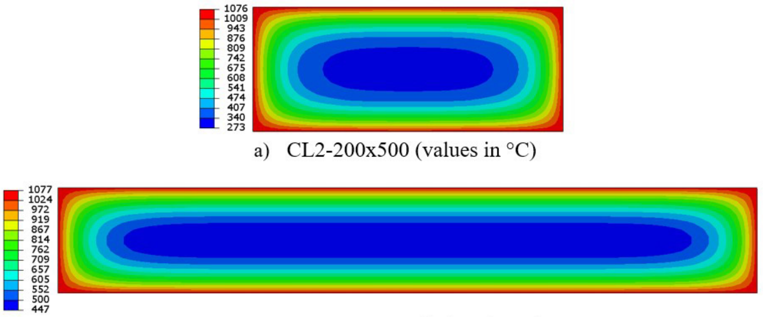
(a) Average temperature of the concrete cross section: the influence of column depth

(b) Average temperature of the concrete cross section: the influence of column width

(c) Average temperature of the steel reinforcement

(d) Average temperature of the steel reinforcement: for axis distance C of 10, 20, 30 and 40 mm (columns 3000 mm of depth only)

Figure 13 – Temperature for columns with four sides fire exposed exposed (fire case V)


a) CL2-200x500 (values in °C)

b) CL9-150x1000 (values in °C)

Figure 14 – Thermal gradient of column cross-section (fire case V) at 120 min

The average temperature of the concrete increases more than 60% compared to the previous case (see Figure 11a and Figure 11b to Fire Case IV and Figure 13a and Figure 13b to Fire Case V). This demonstrates that the heating of the longer side of the column prevails in the thermal results, as already discussed in the comparison between the Fire Case I to IV.

Increasing the "W" by 50 mm decreases the average temperature between 30-50 °C (Figure 13b). Figure 14 shows that when four sides are heated, no part of the cross section is at room temperature. At the end of 120 min in W=150 mm, the lowest temperature of the cross section is 447 °C, while heating the column from three sides (i.e., fire case IV) is 105 °C. The average temperature of the rebars (in columns of the same "W" and with a C=30 mm) does not change with increasing column depth (Figure 13b). The exception is columns of smaller width (150 mm), where the thermal gradient of the section tends to increase slightly (Figure 13b and Figure 14c). The variable that has a greater effect on rebars temperatures is "C" (Figure 13d).

Compared to a column heated from two or three surfaces, the temperature profile in the reinforcements increases by 200 and 300 °C, respectively, in columns heated from all surfaces. In this case, all the reinforcements located around the perimeter of the column are heated. There is a reduction in the mechanical properties of the concrete and the reinforcement, which can be of the order of 30% or 70% (in the case of C=30 mm), respectively, at the end of the 120 min.

The numerical results also demonstrated that columns and shear walls subjected to fire for all surfaces showed very similar results in relation to the EN 1992-1.2. The critical 500°C of the concrete was reached in the case of the columns, but not in the case of the shear walls. Using C=20 mm it is not possible to achieve FRR=60 min, but with C=30 it is possible to achieve FRR=90 min. These results converge with Method A proposed by EN 1992-1.2.

The results of FE also show that a shear wall with a width W of 300 mm and C of 25 and 35 can achieve an FRR of 90 and 120 min, respectively. The same is true for the shear wall with W=400 mm and C=40 mm, which achieves FRR=90 min, which is in agreement with the standardized Method A. These results clearly show that the standard tabulated procedures (assuming that shear walls are heated more than one side) refer to structures that are heated from all surfaces. The EN 1992-1.2 becomes conservative when it is assumed that the column is heated from two or three sides.

3.2.6 Correlation between Fire case I to V

Figures 15 and 16 show the correlation between fire cases I to V. In Figure 15, for example, column CL1-200x300 FCI, CL1-200x300 FC II, CL1-200x300 FC III, CL1-200x300 FC IV and CL1-200x300 FC V refer to column #1 (with a cross-section of 200 x 300 mm, according to Table 1) assuming fire cases (FC) I, II, III, IV and V, respectively. This criterion was used to all results proposed in Figure 15. In Figure 16, for example, the curves D/W=1.5 FC I, D/W=1.5 FC II, D/W=1.5 FC III, D/W=1.5 FC IV and D/W=1.5 FC V refer to the columns with a depth to width ratio (D/W) of a column-shear wall of 1.5, assuming the fire case (FC) I, II, III, IV and V respectively.

The variation of temperatures in concrete and reinforcement during exposure to the ISO 834 is shown in Figure 15. For the temperature in the concrete cross-section, Figure 15a shows the influence of column depth for each fire case and in Figure 15c the influence of width. The same occurs for the reinforcements (Figure 15c and d). Figure 15e and Figure 15f shows for each "C" and each fire case the variation of temperatures in the reinforcement for two cases: 200x3000 and 400x3000 mm.

Figure 15a and b show that heating the column from four sides produces the most severe heating condition. In terms of critical condition, the fire cases can be classified as follows: FC II, FC I, FC III, FC IV, and FC V, where FC II (i.e., one side exposed to fire, the smallest) is the least critical and FC V (all sides exposed to fire) is the most critical. As shown in Figure 15a, assuming the same width, depths greater than 1000 mm have no effect on the average temperature of the cross section. The greater the number of sides subjected to heating, the greater the temperature of the cross-section. In the case of columns heated by only one surface, the longer is more critical than the shorter.

For the same column depth, Figure 15b shows that the heating of the shortest side is not influenced by the column width. On the other hand, the width of the column will influence its average temperature when it is heated from the longest side. In the case of rebars, Figure 15c to f shows that the column's cross-sectional geometry does not influence its average temperature. The main factor is the number of heated sides and the concrete cover values.

Figure 16a shows the temperature in cross-section from the perspective of the D/W ratio of the columns for each Fire Case. The same is shown in Figure 16b, but using columns with the same D/W ratio (2.5 and 7.5 only) with different cross-sectional dimensions. For the rebars, the equivalent is shown in Figure 16c and d. As shown in Figure 16a and b, the D/W cannot be assumed as a variable that influences the fire behavior of the columns. By increasing the D/W with decreasing column width, its average temperature can be higher than a column with higher W and lower D/W.

As expected, Figure 16c and d show that the D/W does not interfere with the average temperature of the reinforcements, since the column cross-section geometry does not influence these results (as previously discussed).

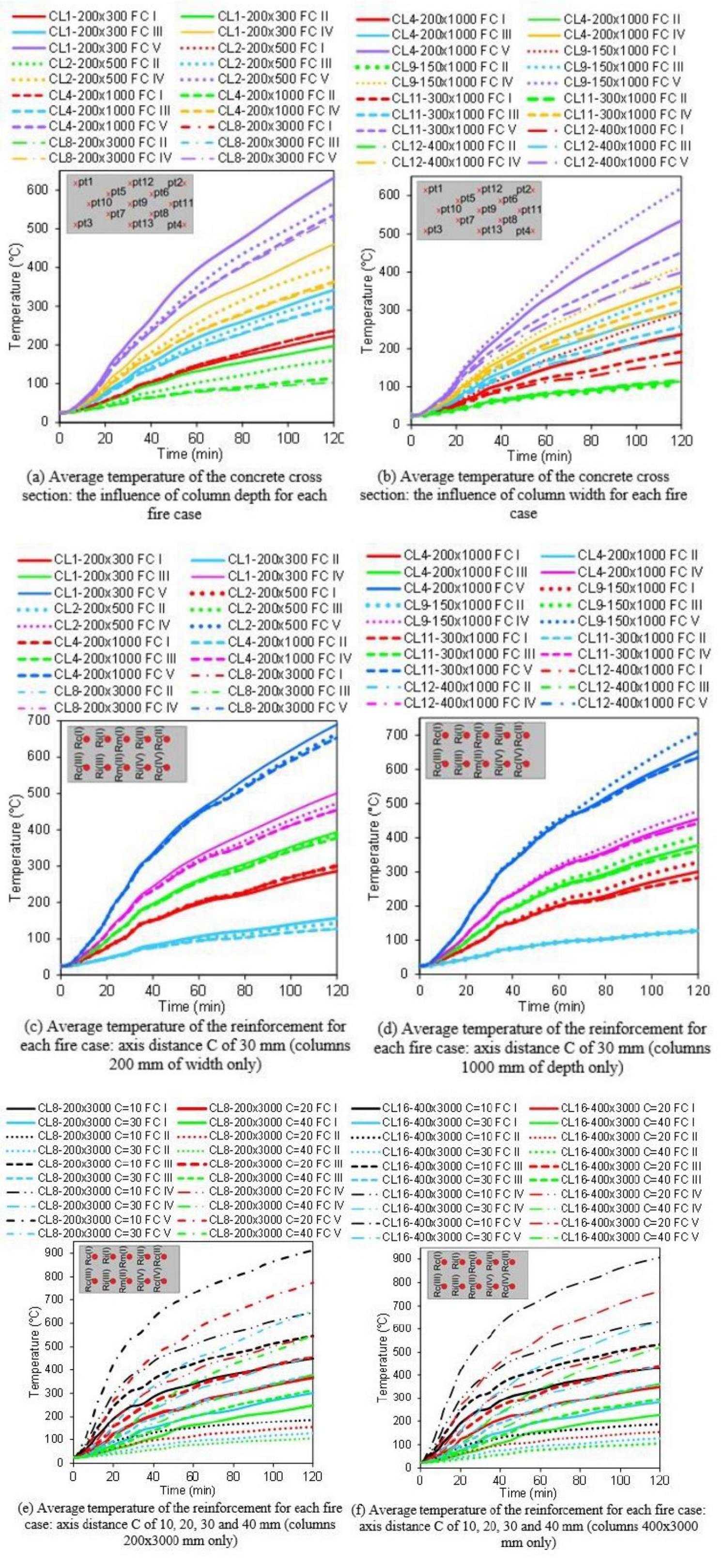


Figure 15 – Temperature for columns (correlation between fire case I to V)

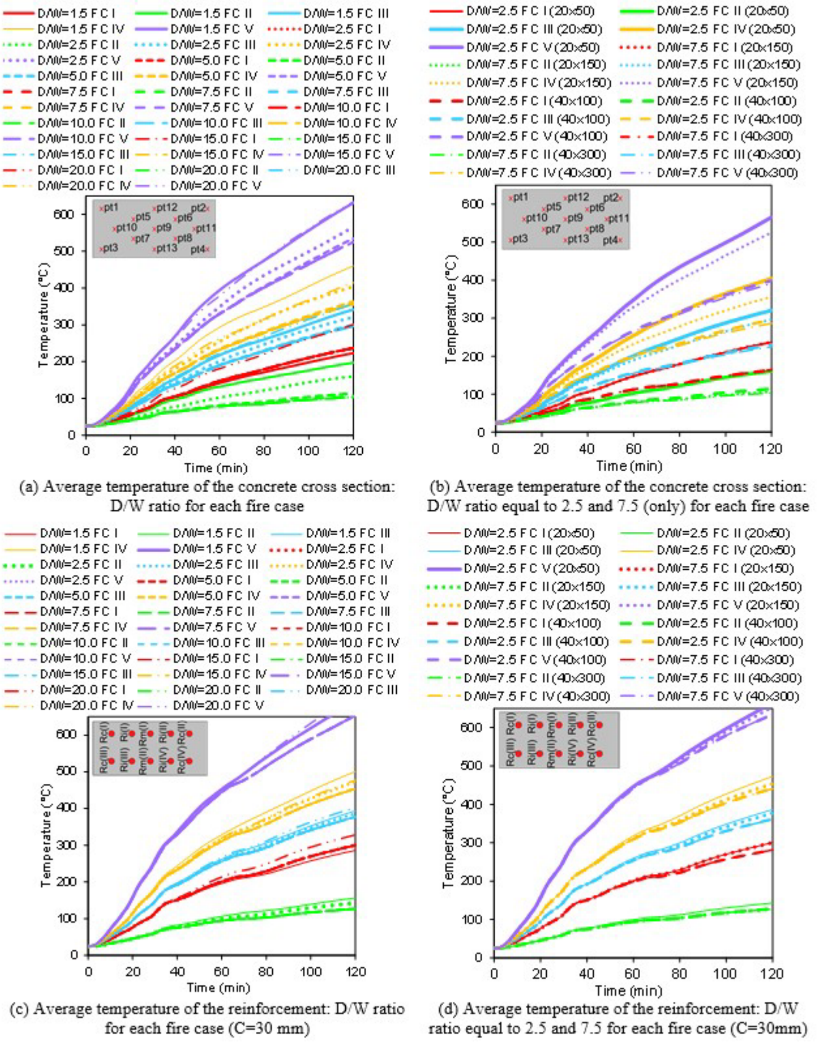


Figure 16 – Temperature for columns (correlation between fire case I to V and D/W ratio)

5 CONCLUSIONS

The following conclusions can be drawn:

- The thermal parameters proposed by EN 1992-1.2 and NBR 15200 showed a good convergence with the experimental results proposed by the authors;
- The number of heated surfaces affects the thermal gradient of shear walls/columns, which influences the mechanical performance of these structures, as temperature causes a decrease (i.e., mechanical damage) of these parameters (tensile strength of steel and compressive strength of concrete) according to EN 1992-1.2 and NBR 15200;
- The EN 1992-1.2 and NBR 15200 suggests two conditions to fire-design columns and shear wall: cross-section (i) exposed to fire by one surface or (ii) by two or more surfaces. This research has shown that considering the exact number of fire exposures surfaces can optimize the results and then the fire design of these structures;
- It was possible to determine fire resistance times that are 120 min higher than those suggested by EN 1992-1.2 and NBR 15200 and to use cross-sections that are not permitted by the standard;
- Depending on the number of heated surfaces, this study has shown that it is possible to use columns or shear walls with W=150 mm, which is not possible according to EN 1992-1.2 and NBR 15200;
- Regarding the shear wall fire exposed by one surface, the EN 1992-1.2 and NBR 15200 proposed a minimum C=25 mm. However, this study shown that columns and shear walls can support at least 120 min with C=25 mm;

- For columns with width of 150 mm exposed to fire on one surface, FRR=90 min was proposed in EN 1992-1.2 and NBR 15200. Based on the thermal gradient of the cross-section, this study has shown that it is possible to achieve an FRR=120 min;
- The column or shear wall's width and C thickness are the main considerations of the standard tabulated procedure. This research demonstrated that the C thickness and the number of heated surfaces have the greatest impact on the FRR results, indicating that the fire-design philosophy of the standard need to be changed;
- The tabulated fire design procedure for columns and shear walls proposed in EN 1992-1.2 and NBR 15200 suggests two hypotheses: heated from one side or from two or more sides. Regarding the first condition, in some cases, FRR higher by 120 min was obtained in this study compared to the standardized. For the second case, the tabular procedures proposed by EN 1992-1.2 show results that converge with the numerical fire case of columns heated from all sides;
- When the column is heated from all sides, the standardized procedure proved to be coherent. However, for cases of columns heated on one, two or three sides, the standardized tabulated method proved to be conservative.

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