CHAPTER VI: SIMPLIFIED METHOD FOR FIRE RESISTANCE OF SELF-COMPACTING CONCRETE FILLED HOLLOW STEEL COLUMNS

Introduction

Concrete filled hollow steel section (HSS) columns in the fire situation can be designed according to Eurocode 4 – Part 1.2, but this document only gives general principles for the temperature analysis and the structural calculation part is tedious to use. A simplified calculation model is provided in Annex H applicable to axially loaded circular or rectangular HSS columns filled with concrete. But the field of application for this method is restricted : percentage of reinforcing steel less than 5 %, buckling length less than 4.5 m.

In order to give to consulting engineers more practical tools, a formula for calculating the fire resistance of SHS columns filled with concrete has been established and is presented in this chapter. Furthermore, the field of applicability has been extended : effective length of column from 2 m to 7 m, percentage of reinforcing steel from 3.5 % to 10 %.

The formula presented here has been established on the basis of simulations performed by the computer code SAFIR.

Five steps have been used :

1. SAFIR results have been compared with experimental results and some calibrations have been performed
2. A formula for short columns with square section has been established based on SAFIR simulations taking into account the main parameters (quality of materials, dimensions, steel bars, concrete cover)
3. It has been shown that the simplified equations can be used for other types of cross-sections
4. The formula has been extended to slender columns
5. An extension is proposed for columns with eccentrical load.

The formulas presented here have been developed on the basis of results for ordinary concrete. It has been shown in this study that the thermal and mechanical properties of self-compacting concrete, when used in hollow sections, are very close to those of normal concrete. Therefore it can be considered that these formulas are also applicable to self-compacting concrete.

VI.1 Fire resistance of columns under centrical load

VI.1.1 Short columns

Using SAFIR programme, the influence of various factors on the fire resistance of very short hollow steel section columns filled with bar-reinforced concrete was investigated through computer-simulated fire tests. These factors include cross-sectional dimensions, quality of materials, steel bar area and concrete cover to the steel bars. A parametrical study has been first performed on square sectional columns. Then, other sectional types have been considered.

In addition to the assumptions described in part III.2.3 and III.3.3, the following assumptions have been implemented for the numerical simulations of short columns:

- Columns are hinged at both ends and subjected to a centrical load, which is constant during the fire;
- The short columns have a length of 0.5 m;
- The short columns are perfectly straight.

VI.1.1.1 Square-sectional columns

The variables that were considered in this research are given below (f_c, f_s and f_y represent the compressive strength of concrete, the yielding stress of reinforcing steel and the yielding stress of structural steel):

- The outside dimension of square HSS columns (D) varies from 150 mm to 300 mm. The wall thickness t varies from 5 mm to 8 mm and D/t is less than $42.3 = 52\sqrt{235/f_y}$ which allows to neglecting the effect of local buckling in the steel wall according to EN 1994-1-1 (2004). The hollow steel sections are: S150*5, S150*6.3, S180*5, S180*6.3, S200*6.3, S220*6.3, S254*6.3, S260*8, S285*8, S300*8. "S" indicates "square" section;
- The amount of steel reinforcement varies from 3.5% to 10%;
- The quality of steel f_y = 355 MPa has been considered in majority because it is now the most commonly used material for hollow steel sections. Some simulations with steel yield strengths f_y = 235 MPa and f_y = 275 MPa are also done to find out the effect of the steel strength on the fire resistance of the columns.
- Three concrete strengths, namely, f_c = 30 MPa, f_c = 40 MPa, f_c = 50 MPa have been used;
- Only the quality of reinforcing steel S500 (f_s = 500 MPa) has been considered in this study because it is now the most commonly used material for reinforcing steel;
- Concrete cover which represents the distance between the axis of longitudinal

reinforcements and the border of the concrete core has been fixed at a minimum value of 30 mm. According to the dimensions of the cross-sections, other concrete covers, namely 35, 40, 45, 50 mm have also been used.

The following symbols have been used in the figures:

N2000 stands for axial load N_{fi} = 2000 KN

S200-6.3 stands for Square section with dimension d = 200 mm, steel wall thickness t = 6.3 mm

C40 stands for compression strength of concrete on cylinder f_c = 40 MPa

Dr45 stands for concrete cover Dr = 45 mm

12D18 stands for the case: reinforcement in the section consists of 12 bars with diameter 18mm

Influence of the load

In the event of a fire, the applied loads are much lower than the maximum design loads specified for normal temperature conditions. This can be explained by two reasons.

- The first one is the fact that the load combination factors γ_s are different under normal conditions and under fire conditions.

  At ordinary temperature, we have:

  $N_{Sd} = (\gamma_s)_{perm}.N_{S\,perm} + (\gamma_s)_{\mathrm{var}}.N_{S\,\mathrm{var}} = \gamma_s^*.N_{S,20°C} = \gamma_s^*(N_{S\,perm} + N_{S\,\mathrm{var}})$

  where:

  N_{Sd} is design acting load

  $N_{S,20°C}$ is total real acting load at ordinary condition

  $(\gamma_s)_{perm}$ is the combination factor for permanent load, $(\gamma_s)_{perm} = 1.35$

  $(\gamma_s)_{\mathrm{var}}$ is the combination factor for variable load, $(\gamma_s)_{\mathrm{var}} = 1.5$

  γ_s^* varies between 1.35 and 1.5 according to the proportion of permanent and variable loads.

  Under fire conditions, we can have $N_{fi} = N_{S\,perm} + N_{S\,\mathrm{var}}$ (if all $\gamma_s = 1$)

  Therefore $\quad N_{fi} / N_{Sd} = \dfrac{1}{\gamma_s^*}$

  Assuming $\gamma_s^* = 1.4$, $N_{fi} / N_{Sd} \approx 0.7$

  That is why values of $N_{fi} / N_{Sd} > 0.7$ are not considered.

- The second one is the fact that different load combinations are used with reduction of variable loads.

  This explain why values much smaller than 0.7 can be found.

In EN 1994-1-2, the term “reduction factor” n_{fi} is the ratio of the design loads on the structures for fire design to the design loads for normal temperature design:

$$n_{fi} = \frac{N_{fi}}{N_d}$$

where N_{fi} is the compression load on the column under fire condition; N_d is the compression load on the column under normal temperature.
The preceding considerations are illustrated are by Figure VI-1.

When $Q_{k,1} = 0$, there are only permanent loads, and $\gamma_s^* = 1.35$; thus $\eta_{fi} = \frac{1}{1.35} = 0.74$.

When $Q_{k,1} / G_k$ becomes large and $\psi_{1,1}$ small, the values of η_{fi} decrease very much.

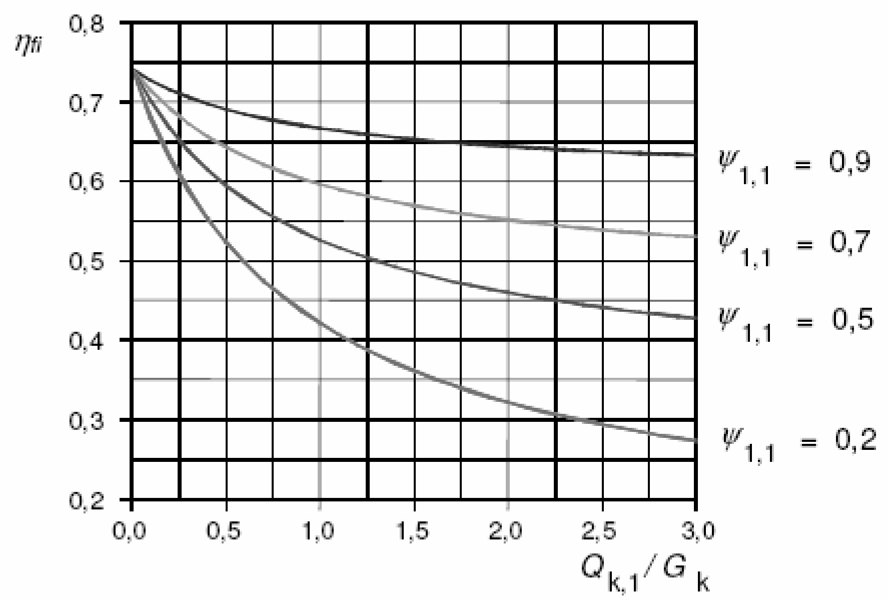


Figure VI-1 Variations of reduction factors n_{fi} with the load ratio $Q_{k,1} / G_k$ (EN 1994-1-2) (where $Q_{k,1}$ is the characteristic value of the leading variable action 1, G_k is the characteristic value of the permanent action, $\psi_{1,1}$ is factor for frequent value of the variable action for fire situation)

The value $n_{fi} \leq 0.65$ is recommended except for imposed loads according to load category E as given in EN 1991-1-1 (areas susceptible to accumulation of goods, including access areas), where the recommended value is 0.7 (EN 1994-1-2 part 2.4.2)

In this part, the parameter $N_{fi} / N_{pl,Rk}$ is considered where $N_{pl,Rk}$ is the characteristic value of the plastic resistance to compression of the section given by:

$$N_{pl,Rk} = A_a . f_y + A_c . f_{ck} + A_s . f_{sk}$$

At ordinary temperature conditions, design is based on: $N_d \leq N_{pl,Rd}$

where $N_{pl,Rd} = A_a . f_{yd} + A_c . f_{cd} + A_s . f_{sd} = A_a . \frac{f_y}{\gamma_a} + A_c . \frac{f_{ck}}{\gamma_c} + A_s . \frac{f_{sk}}{\gamma_s}$

$\gamma_a = 1.0$ (steel of the tube)

$\gamma_c = 1.5$ (concrete)

$$\gamma_s = 1.15 \qquad \text{(reinforcing steel)}$$

We can consider $N_{pl,Rd} = \dfrac{N_{pl,Rk}}{\gamma_m^*}$ with γ_m^* varying between 1.0 and 1.5 according to the proportions of the materials.

Let us consider a mean value of $\gamma_m^* = 1.2$. We assume also $N_d = N_{pl,Rd}$

$$\text{If } \frac{N_{fi}}{N_{pl,Rd}} = \frac{N_{fi}}{N_d} = 0.7 \text{ and } \gamma_m^* = \frac{N_{pl,Rk}}{N_{pl,Rd}} = 1.2$$

$$\text{then } \frac{N_{fi}}{N_{pl,Rk}} = \frac{0.7}{1.2} \approx 0.6$$

And the upper limit of $\dfrac{N_{fi}}{N_{pl,Rk}}$ is approximately 0.6.

Therefore, in this study, the range $0.2 \le N_{fi} / N_{pl,Rk} \le 0.6$ is considered only.

The calculated fire resistance is plotted as a function of the axial load for various values of concrete cover in Figure VI-2.

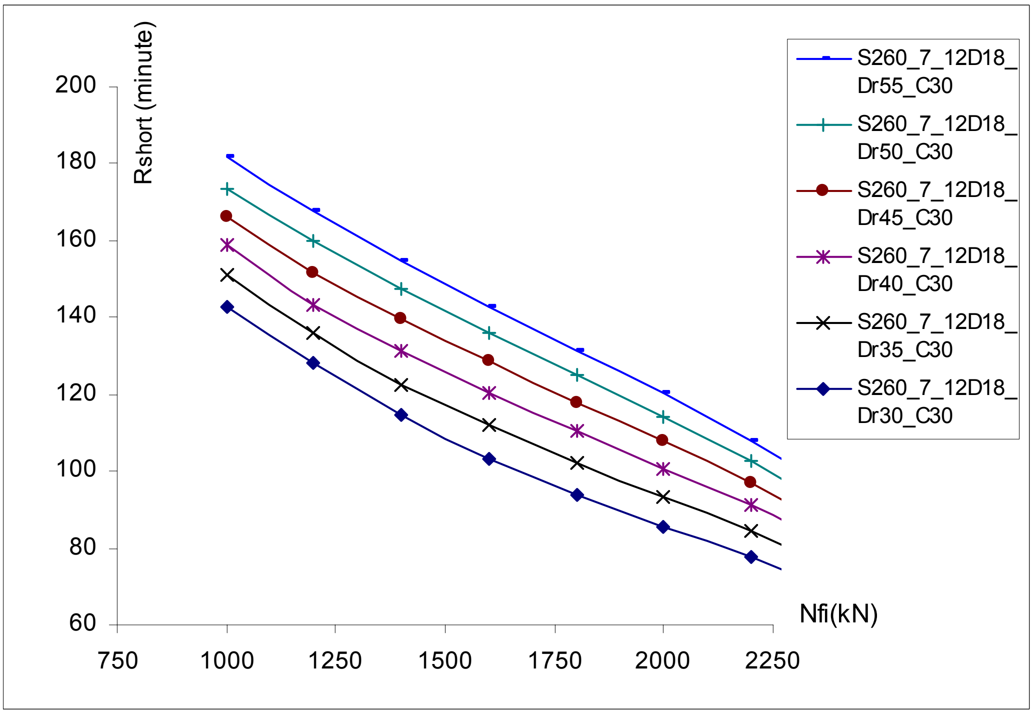


Figure VI-2 Correlation between R_{short} and N_{fi} for various values of concrete cover

The influence of the load on the fire resistance of CFSHS columns has been also investigated for various cross sections and concrete strengths. The fire resistance is plotted as a function of the axial load for various other parameters (Figure VI-3). The relationship between fire resistance and load can be expressed as $R_{short} = a * \sqrt{N_{fi}} + b$ whatever the cross section dimension, the steel bar area and concrete strength (the linearity can be seen clearly on Figure VI-4).

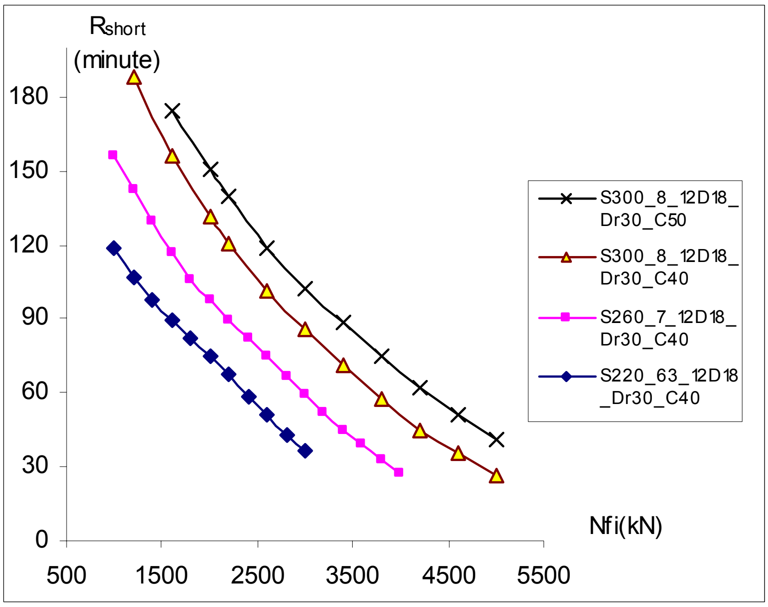


Figure VI-3 Fire resistance as a function of load for various cross sections and concrete strengths

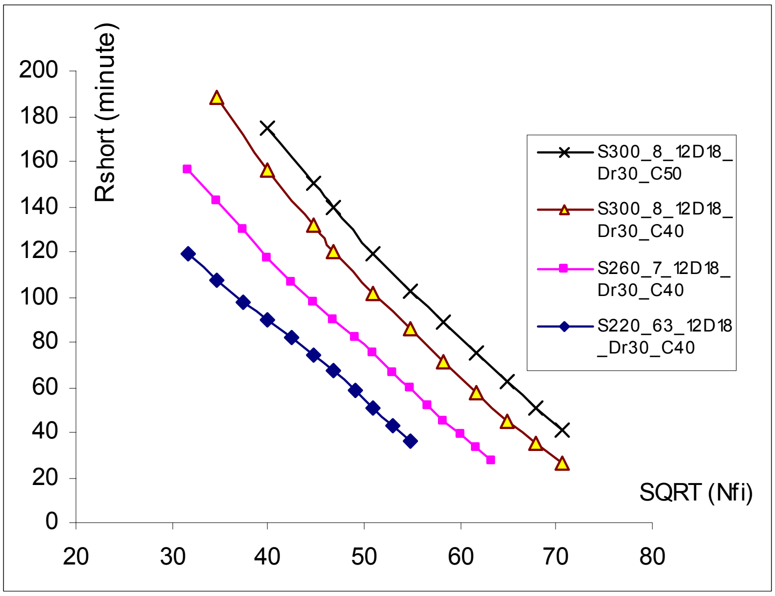


Figure VI-4 Fire resistance as a function of $\sqrt{N_{fi}}$ for various cross sections and concrete strengths

Influence of the concrete area and strength

The influence of the concrete area and strength on the fire resistance of HSS columns has been studied by finding the relationship between the fire resistance and $N_c = A_c * f_c$. In Figure VI-5 the fire resistance is plotted as a function of N_c for various other parameters. It can be seen that the relationship between fire resistance and load is linear whatever the cross section dimensions, the steel bar area, the concrete strength and the concrete cover.

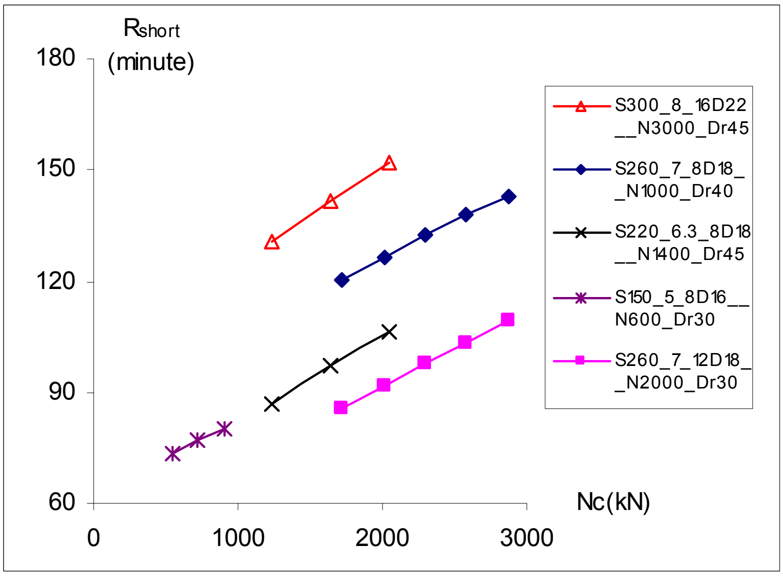


Figure VI-5 Fire resistance as a function of N_c for various cross-section dimensions, loads and concrete covers

Influence of the steel bar area

The influence of the steel bar area on the fire resistance of CFSHS columns has been studied by finding the relationship between the fire resistance and $N_s = A_s * f_s$. In Figure VI-6 the fire resistance is plotted as a function of N_s for various values of concrete cover. It can be seen that the relationship between fire resistance and steel bar area is a curve affected by the value of concrete cover.

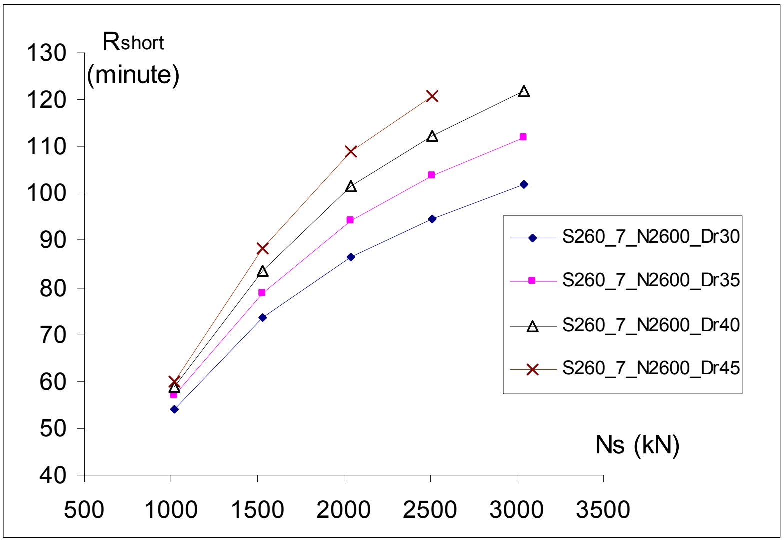


Figure VI-6 Fire resistance as a function of N_s for various concrete covers

Influence of the concrete cover

An increase of concrete cover increases the fire resistance of short columns as can be seen in Figure VI-7

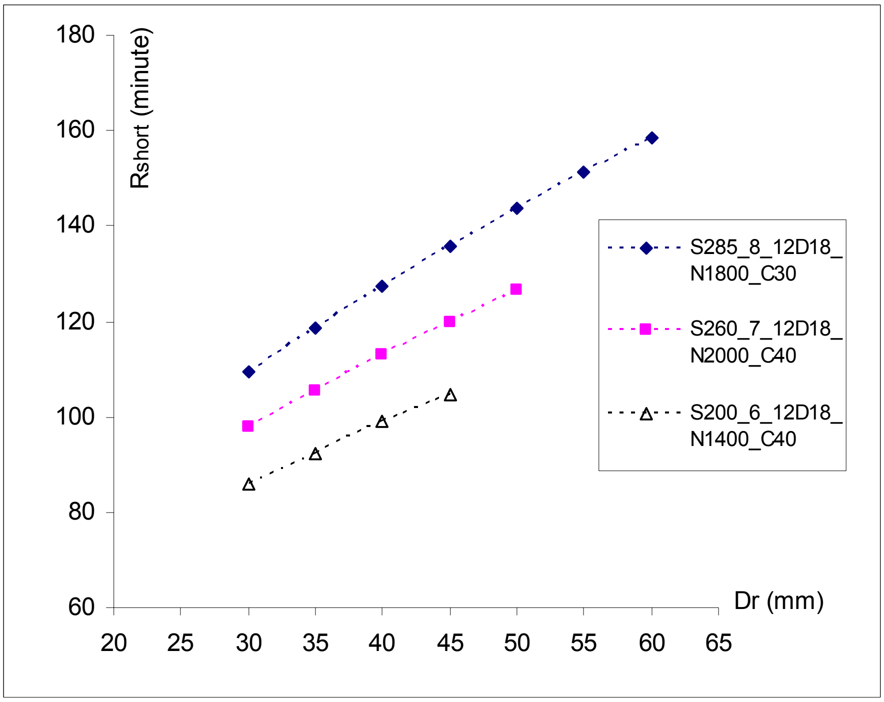


Figure VI-7 Fire resistance as a function of Dr for various loads, concrete strengths and cross-sectional dimensions

Influence of steel wall thickness

To investigate the influence of steel wall thickness on the fire resistance of the column, for some cross-sections S150*5, S200*6.3, S260*8, S300*8, the concrete core is kept unchanged, the steel wall thickness varies from 4mm to 12mm, the steel wall strength is f_y =355 MPa. The correlations between the fire resistance of short columns and the steel wall thickness are presented for various values of the cross section, applied load, concrete cover, concrete strength, reinforcement area (Figure VI-8).

It can be seen that the influence of steel wall thickness t on the fire resistance of short columns R_{short} is linear with a small slope, whatever the other parameters. So the fire resistance as a function of steel wall thickness can be expressed as $R_{short} = a * t + b$, where a, b are independent from steel wall thickness.

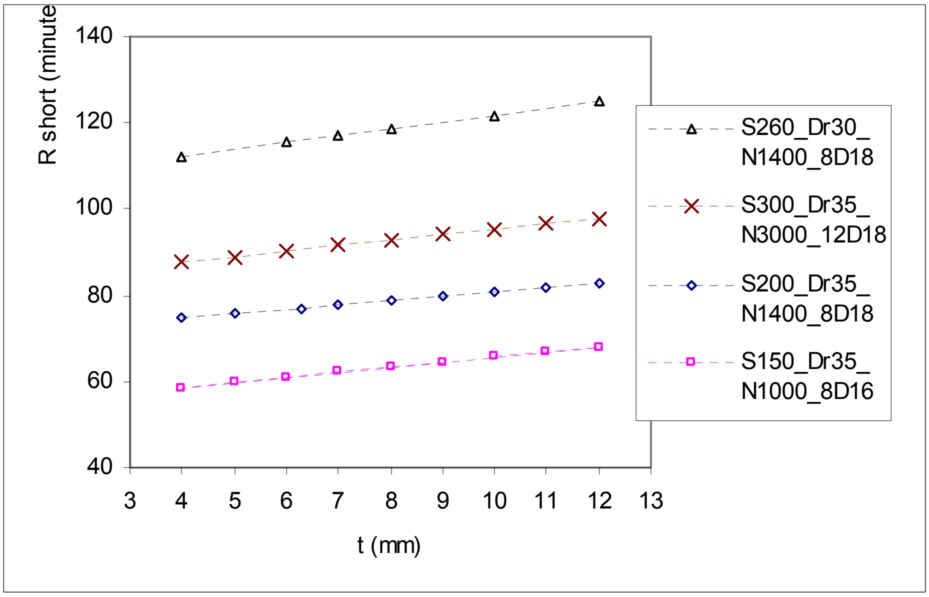


Figure VI-8 Fire resistance as a function of steel wall thickness

Influence of steel wall strength

The fire resistances of short columns are calculated for three values of steel strength: $f_y = 235$ MPa, $f_y = 275$ MPa, and $f_y = 355$ MPa.

Table VI.1 shows an example of the influence of the steel wall strength on R_{short} of square section dimension of 200 mm. Various values of steel wall thickness and applied load have been considered. It can be seen that the fire resistance increases slightly with the steel strength. For columns with a fire resistance larger than 30 minutes, the fire resistance increase due to the increase of steel wall strength becomes negligible. This can be explained by the fact that after 30 minutes in fire, the temperature in the external steel wall is above 700°C (Figure VI-9). At that high temperature, steel loses almost completely its strength and stiffness. Therefore the steel grade affects little the fire resistance. In this study, the quality of steel f_y=355 MPa has been considered in majority because it is now the most commonly used material for hollow steel sections. For other steel grades, the formula built for f_y=355 MPa can also be used.

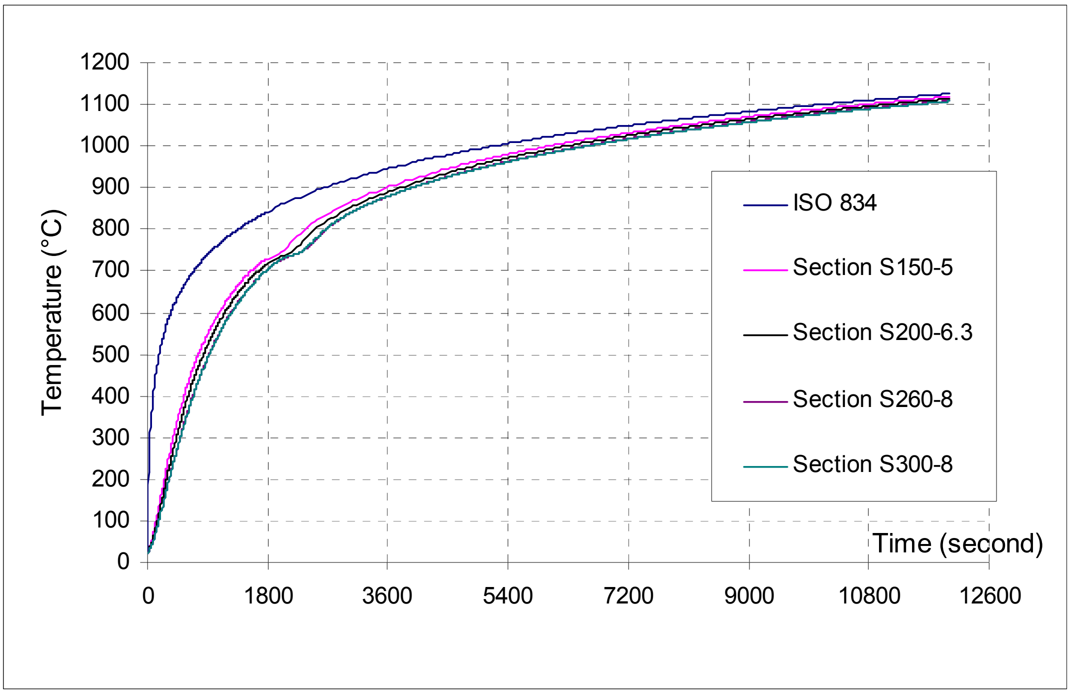


Figure VI-9 Temperature of steel wall varies with time exposed to fire

Steel thickne	Loading	Rshort with fy=355 MPa	Rshort with fy=275 MPa		Rshort with fy=235 MPa	
mm	Nfi(kN)	R_355 (minute)	R_275 (minute)	R_275/R_355	R_235 (minute)	R_235/R_355
4.0	2800	13.98	11.81	0.85	9.81	0.70
5.0	2800	16.67	14.63	0.88	13.33	0.80
5.0	2600	20.25	18.08	0.89	16.96	0.84
6.3	2600	23.33	21.15	0.91	19.81	0.85
5.0	2400	25.25	23.29	0.92	22.21	0.88
9.0	2600	28.65	26.33	0.92	25.04	0.87
10.0	2600	30.77	28.04	0.91	26.71	0.87
4.0	2200	31.96	29.94	0.94	28.83	0.90
7.0	2200	38.98	37.08	0.95	35.85	0.92
9.0	2200	42.71	40.79	0.96	39.58	0.93
6.3	2000	46.73	45.00	0.96	44.02	0.94
9.0	2000	51.83	49.58	0.96	48.29	0.93
5.0	1800	55.79	54.81	0.98	54.29	0.97
6.3	1800	57.69	56.54	0.98	55.94	0.97
7.0	1800	58.69	57.44	0.98	56.81	0.97
8.0	1800	60.10	58.71	0.98	58.00	0.96
9.0	1800	61.52	60.00	0.98	59.17	0.96
10.0	1800	62.92	61.21	0.97	60.33	0.96
11.0	1800	64.33	62.44	0.97	61.48	0.96
4.0	1600	64.71	63.96	0.99	63.63	0.98
12.0	1800	65.83	63.67	0.97	62.65	0.95
5.0	1600	66.21	65.27	0.99	64.79	0.98
6.3	1600	68.10	66.94	0.98	66.35	0.97
7.0	1600	69.13	67.83	0.98	67.17	0.97
8.0	1600	70.58	69.10	0.98	68.35	0.97
9.0	1600	72.04	70.38	0.98	69.54	0.97
10.0	1600	73.50	71.65	0.97	70.71	0.96
11.0	1600	74.94	72.90	0.97	71.88	0.96
4.0	1400	75.52	74.94	0.99	74.65	0.99
12.0	1600	76.33	74.17	0.97	73.08	0.96
5.0	1400	76.79	76.06	0.99	75.71	0.99
6.3	1400	78.42	77.52	0.99	77.06	0.98
7.0	1400	79.29	78.29	0.99	77.79	0.98
8.0	1400	80.54	79.40	0.99	78.83	0.98
9.0	1400	81.77	80.50	0.98	79.85	0.98
10.0	1400	83.08	81.58	0.98	80.88	0.97
11.0	1400	84.23	82.69	0.98	81.90	0.97
12.0	1400	85.46	83.77	0.98	82.92	0.97
4.0	1200	85.52	84.94	0.99	84.65	0.99
5.0	1200	86.77	86.04	0.99	85.69	0.99
6.3	1200	88.40	87.48	0.99	87.02	0.98
7.0	1200	89.25	88.25	0.99	87.75	0.98
8.0	1200	90.48	89.35	0.99	88.77	0.98

Table VI.1 Comparison of the fire resistance (Rshort) of the columns with various steel wall strengths

Formula for calculating the fire resistance of short columns

Based on the data from parametric studies, a formula has been developed for the calculation of the fire resistance of square steel hollow section columns filled with bar-reinforced concrete. The proposed formula is:

for $0.2 \le N_{fi} / N_{pl,Rk} \le 0.6$:

$$R_{short} = a_1 + a_2 * t + a_3 * N_c + a_4 * \sqrt{N_{fi}} + a_5 * \sqrt{N_s \times D_r} + a_6 * d \qquad \text{(VI.1)}$$

where a_1, a_2, a_3, a_4, a_5, and a_6 are constant; *t*, *Dr* and *d* in mm ; N_c, N_{fi} and N_s in KN

With more than 7500 numerical calculations concerning almost all cases and using MATLAB program, regression coefficients a_1, a_2, a_3, a_4, a_5, and a_6 have been computed by performing a least square fit. The following values have been obtained:

a_1 = 50 (minute) a_2 = 0.33 (minute/mm) a_3 = 0.018 (minute/KN)

a_4 = -3.93 (minute/$\sqrt{KN}$) a_5 = 0.33 (minute/$\sqrt{KN.mm}$) a_6 = 0.42 (minute/mm)

A comparison between the results obtained from the numerical model and the simplified method is shown in Figure VI-10.

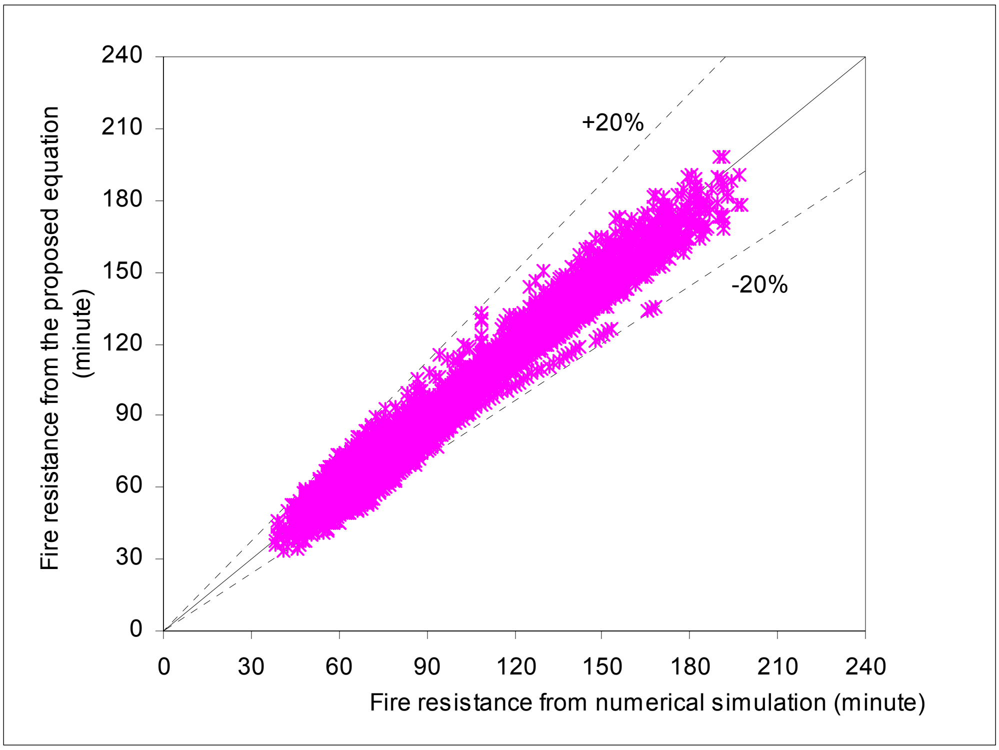


Figure VI-10 Comparison of fire resistance of columns from Eq.(VI.1) with predictions of the numerical model with $0.2 \leq N_{fi} / N_{pl,Rk} \leq 0.6$

To check the errors, the correlations between Rsimple/Rnumeric and various factors are shown in Figure VI-11 to Figure VI-15. Rsimple is the fire resistance calculated from equation(VI.1), Rnumeric is the fire resistance calculated from the numerical model. Figure VI-11 to Figure VI-15 show that there is no systematic deviation.

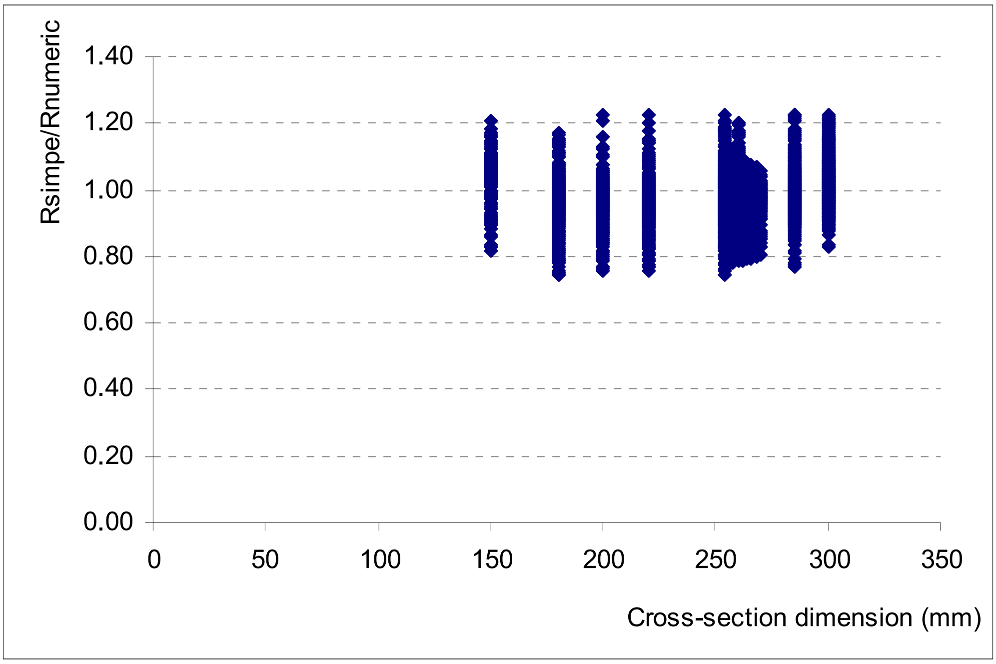


Figure VI-11 Correlation between Rsimple / Rnumeric and cross-section dimension

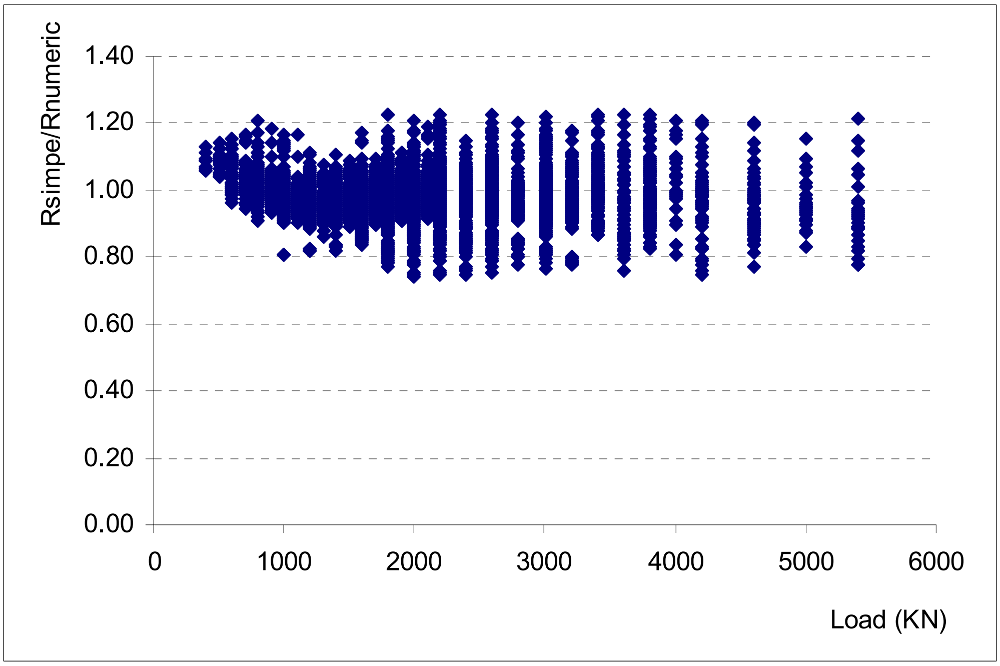


Figure VI-12 Correlation between Rsimple / Rnumeric and load

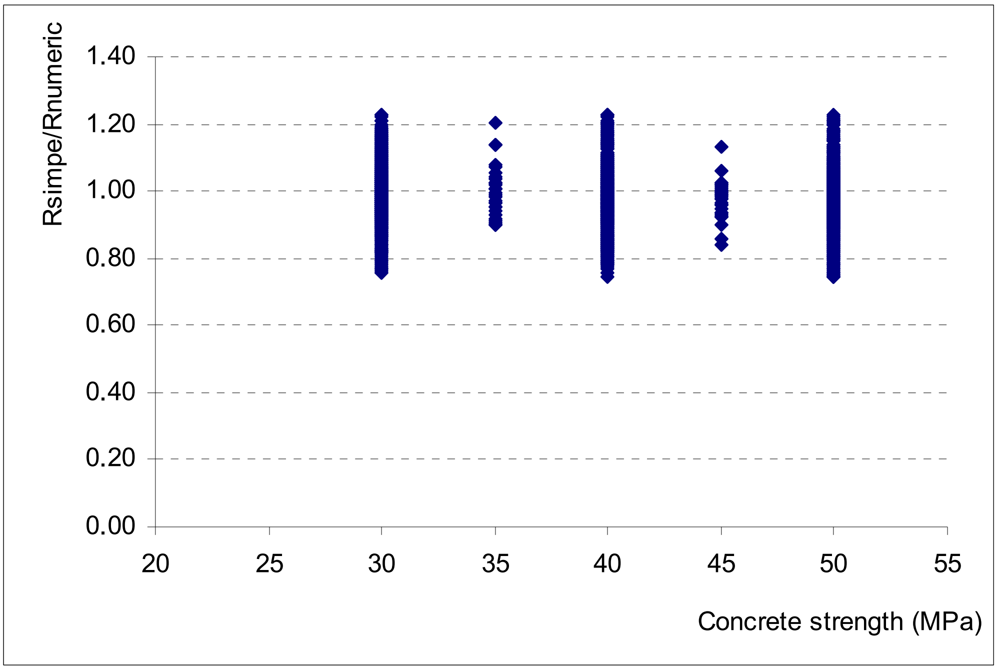


Figure VI-13 Correlation between Rsimple / Rnumeric and concrete strength

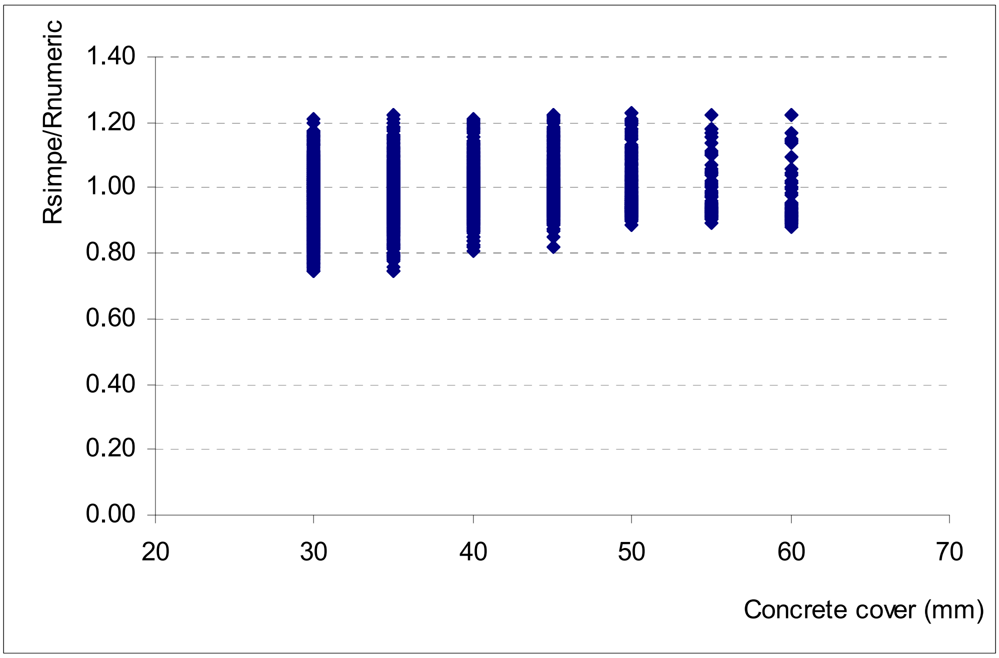


Figure VI-14 Correlation between Rsimple / Rnumeric and concrete cover

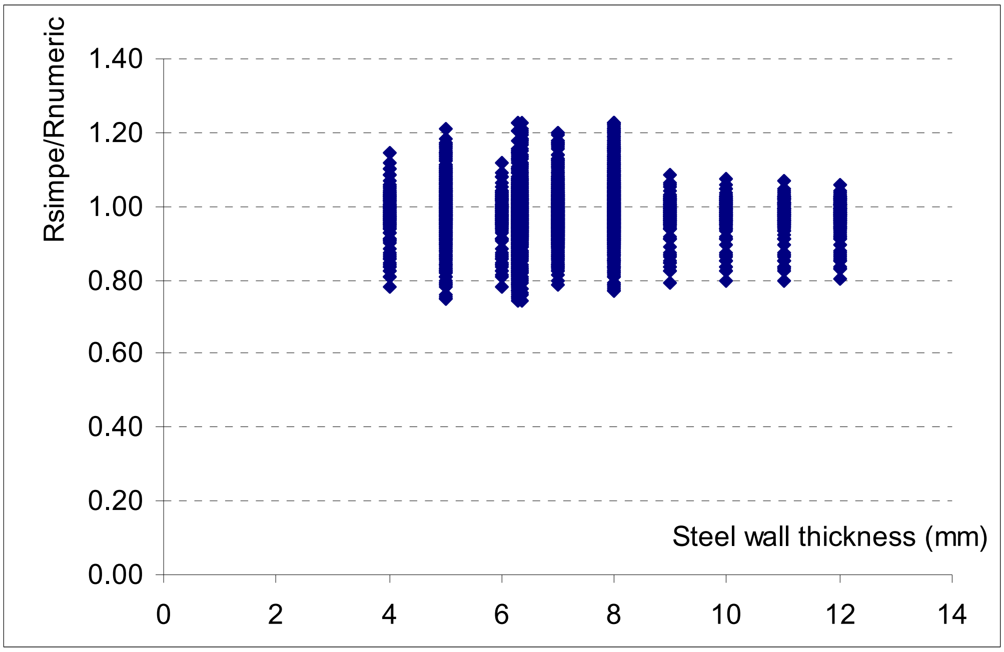


Figure VI-15 Correlation between Rsimple / Rnumeric and steel wall thickness

From regression equation(VI.1), it can be seen that the steel wall thickness *t* contributes very little to the fire resistance of short columns (increasing the thickness t by 1mm give an increase of only 0.33 minute of the fire resistance). One can thus eliminate *t* in equation(VI.1) and create equation (VI.2) by assuming t = 6 mm in all cases.

$$R_{short} = 52 + 0.018 * N_c - 3.93\sqrt{N_{fi}} + 0.33*\sqrt{N_s \times D_r} + 0.42 * d \qquad \text{(VI.2)}$$

In the reference Renaud C. (2004), the influence of hollow steel thickness on the column buckling coefficient has been investigated using the numerical model SISMEF. Results also show that the increase of hollow steel section thickness seems to have a small effect on the column behaviour.

VI.1.1.2 Other types of column cross-sections.

Circular cross-section

Calculations of fire resistance of circular short columns have been performed. The diameter of the section varies from 168.3 mm to 323.9 mm. The calculations have been made with three values of concrete strength (C30, C40 and C50) and reinforcement ratio from 3.5% to 10%. Results show that the simplified equation (VI.1) can still be used for circular cross-section columns (Figure VI-16). A_s is now taken as the section of the embedded steel profile and the concrete cover is considered on the embedded steel profile.

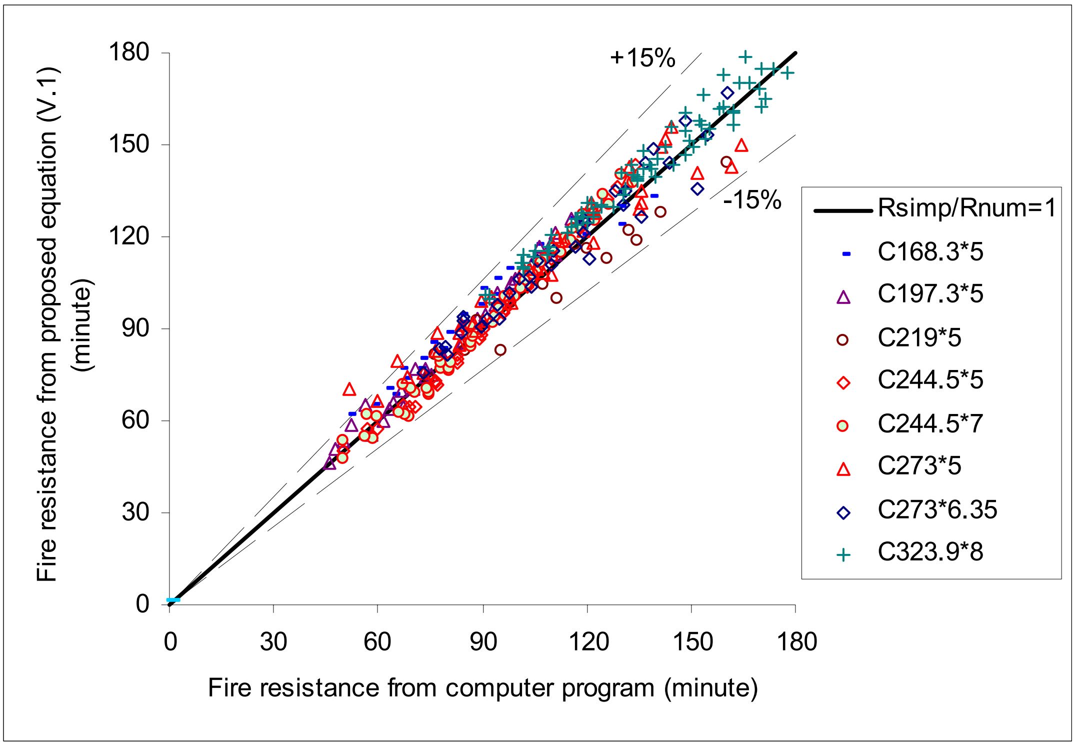


Figure VI-16 Comparison of fire resistance of circular sectional columns from Eq.(VI.1) with predictions of the numerical model

Cross-section with embedded steel profile

Calculations of fire resistance of short columns with embedded steel profile shown in figure VI-19 have been performed.

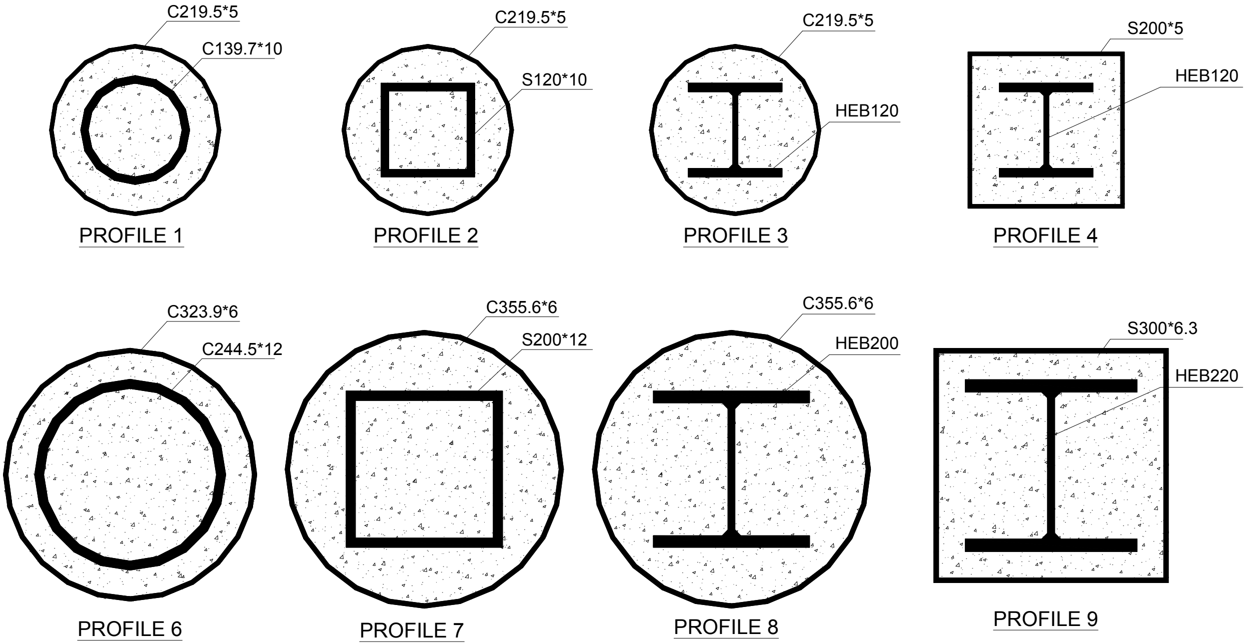


Figure VI-17 Cross-section with embedded steel profile

Results show that the simplified equation (VI.1) can still be used for this type of cross-section. (Figure VI-18)

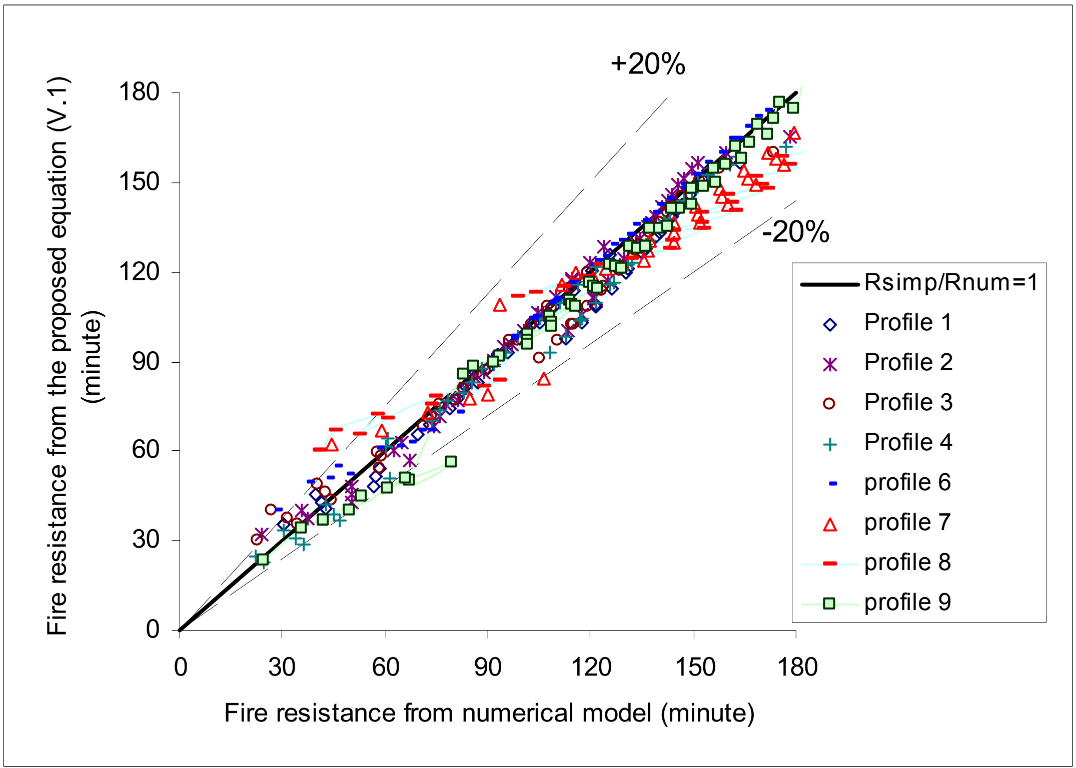


Figure VI-18 Comparison of fire resistance of embedded steel profile columns from Eq.(VI.1) with predictions of the numerical model

Cross-section with double skin tubular steel profiles

Comparison between the fire resistance obtained from the numerical model and the simplified equations (VI.1) was also made for short columns of double skin tubular steel profiles (Figure VI-19). Results show that in case of a section with a void, the simplified equations give a fire resistance larger than that calculated from the numerical model.

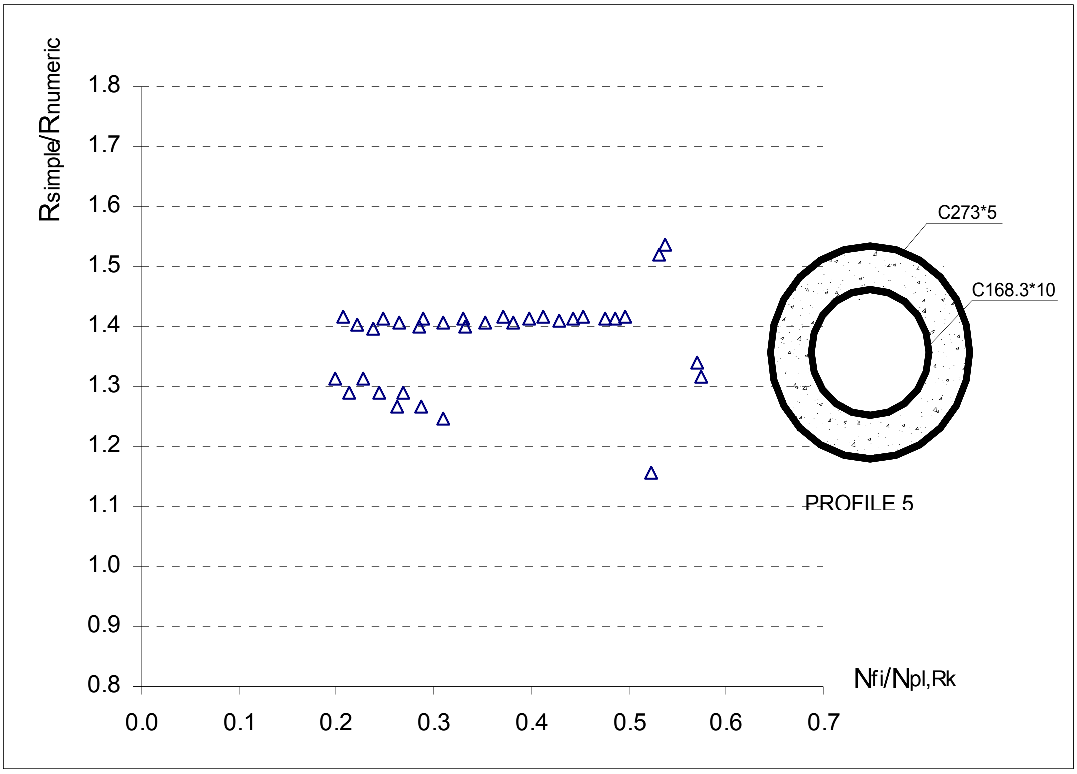


Figure VI-19 Correlations between Rsimple / Rnumeric and $N_{fi} / N_{pl,Rk}$

VI.1.2 Slender columns

The fire resistance of short columns can be calculated from Eq.(VI.1). Now, SAFIR program is used for calculating the fire resistance of columns with various lengths and finding out the influence of column length on the fire resistance.

The assumptions of the numerical simulations are described in part III.2.3 and III.3.2

VI.1.2.1 Influence of column length on the fire resistance

The correlation between the fire resistance of slender columns and column length is shown in Figure VI-20. It can be seen that the relationship between fire resistance and column length is almost linear except for some ranges corresponding to very small values of the fire resistance (never used in practice).

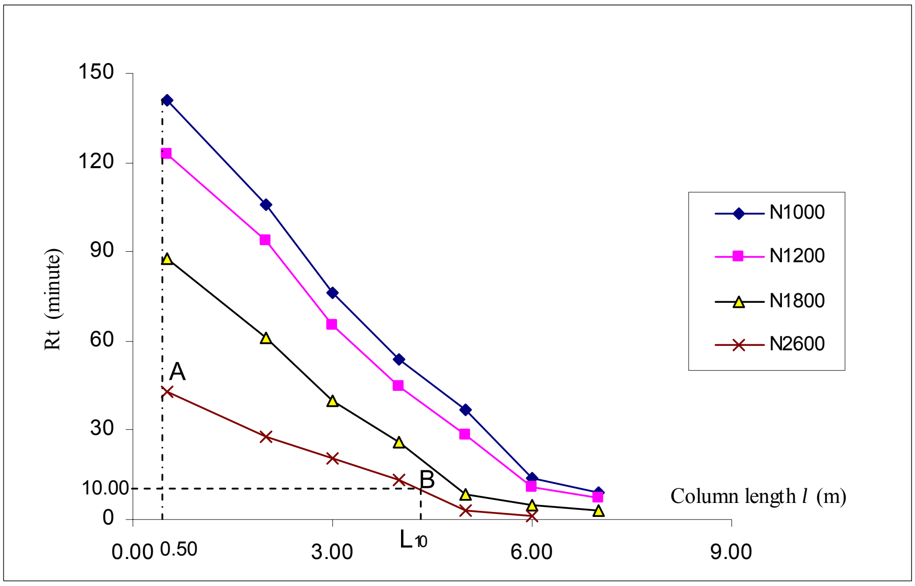


Figure VI-20 Fire resistance as a function of column length of square sectional columns

VI.1.2.2 Relationship between fire resistances of short and slender columns

In the studied range, it can be seen that the fire resistance of a column with a given length can be linearly interpolated between two points: point A (Figure VI-20) corresponds to the fire resistance of very short column (with a length of 0.5 m), point B corresponds to a slender column with a fire resistance of 10 minutes. Let's call the length corresponding to a fire resistance of 10 minutes L_{10}.

The proposed formula is:

$$R_f = R_{short} - \frac{R_{short} - 10}{L_{10} - 0.5}(l\text{-}0.5) \quad \text{(VI.3)}$$

with R_f and R_{short} in minutes ; L_{10} and l in meters

R_{short} is the fire resistance of short column

R_f is the fire resistance of slender column

L is the effective length of the column.

It is now needed to find L_{10}. To this aim the following considerations can be made. After 10 minutes of fire, with sectional dimensions from 150 mm to 350 mm and steel wall thicknesses from 5mm to 8mm, the temperatures in reinforcing steel and concrete are below 100°C, the temperatures in steel wall are approximately 400°C (Figure VI-21). Therefore the mechanical properties of concrete and reinforcing steel are unchanged compare to those at room temperatures. The strength of the steel wall is unchanged but the modulus of steel wall at 400°C is about 70% of that at room temperature. The value L_{10} can therefore be obtained using the method for analysing axially loaded composite columns at normal temperature. But the effective flexural stiffness of the cross section should be reduced because of the cracks in the concrete and the yielding of the steel.

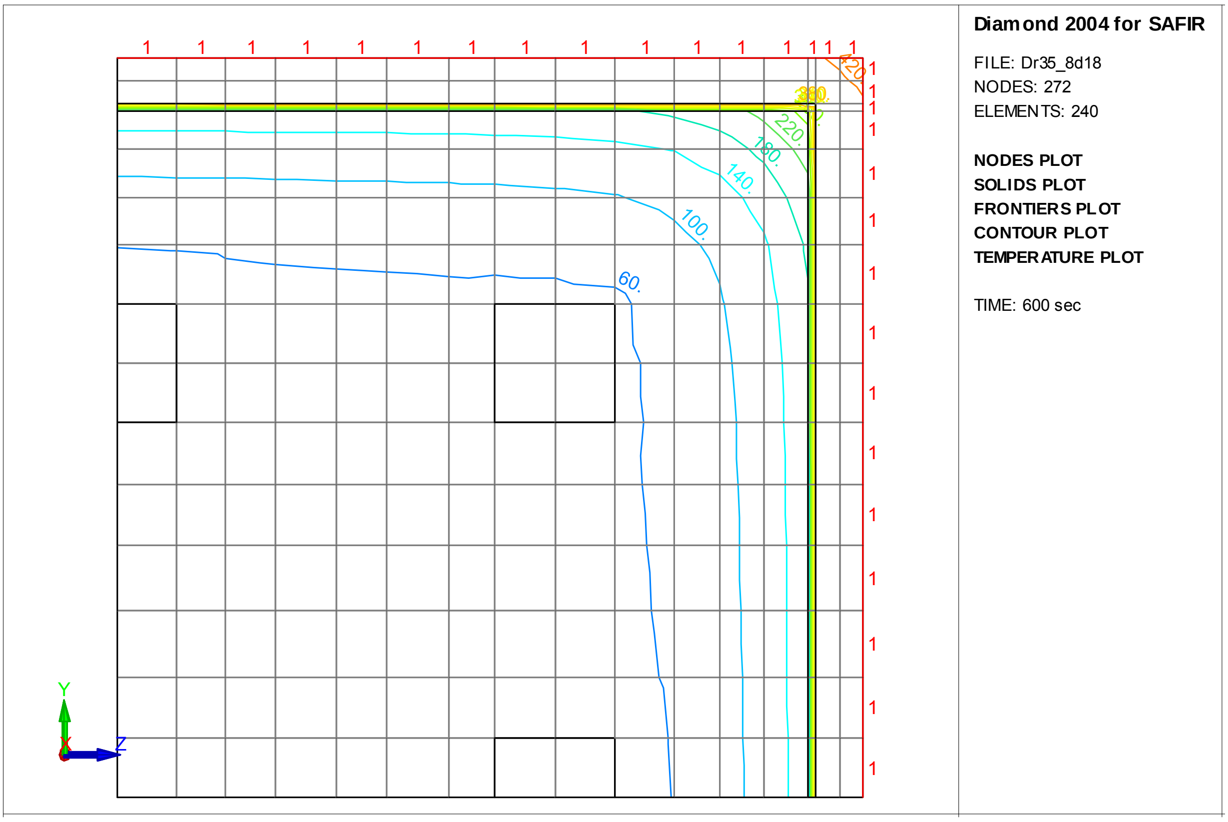


Figure VI-21 Temperature distribution in a section S200_6.3 after 10 minutes of fire

Calculation of the value L_{10}

To calculate the value L_{10}, relationships between load bearing capacities of the columns and the lengths for fire duration R10 (the fire resistance is 10 minutes) were performed. Columns with various concrete strengths (30 MPa, 40 MPa and 50 MPa), reinforcement ratio (3.5% to 10%) and cross-section dimensions (from 150mm to 300 mm) have been calculated. Some cases are shown from Figure VI-22 to Figure VI-25.

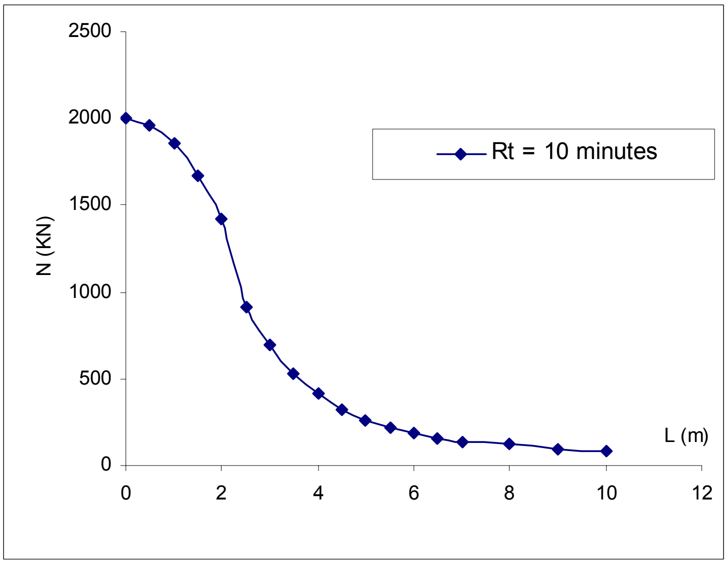


Figure VI-22 Buckling curve of the column with section S150_5_4D16_Dr30_C40 for fire duration R10

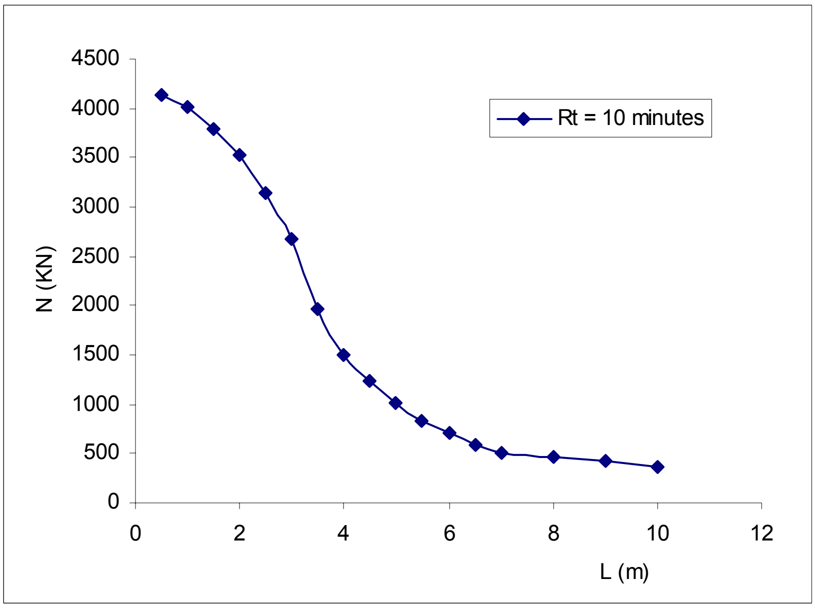


Figure VI-23 Buckling curve of the column with section S200_6.3_8D18_Dr30_C50 for fire duration R10

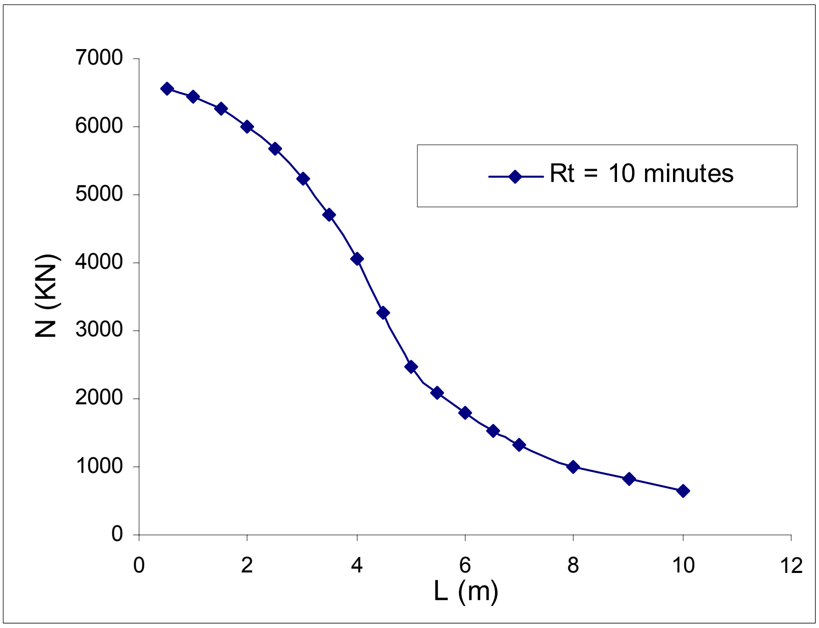


Figure VI-24 Buckling curve of the column with section S260_8_12D18_Dr45_C50 for fire duration R10

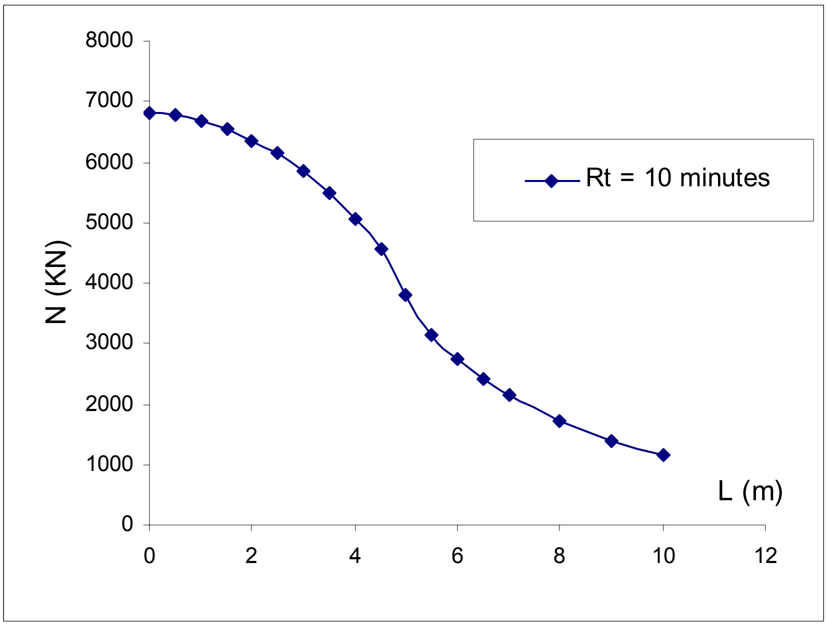


Figure VI-25 Buckling curve of the column with section S300_8_12D18_Dr30_C30 for fire duration R10

It can be seen that the shape of the curves is similar to curve "*a*" in European buckling curves (Figure VI-26).

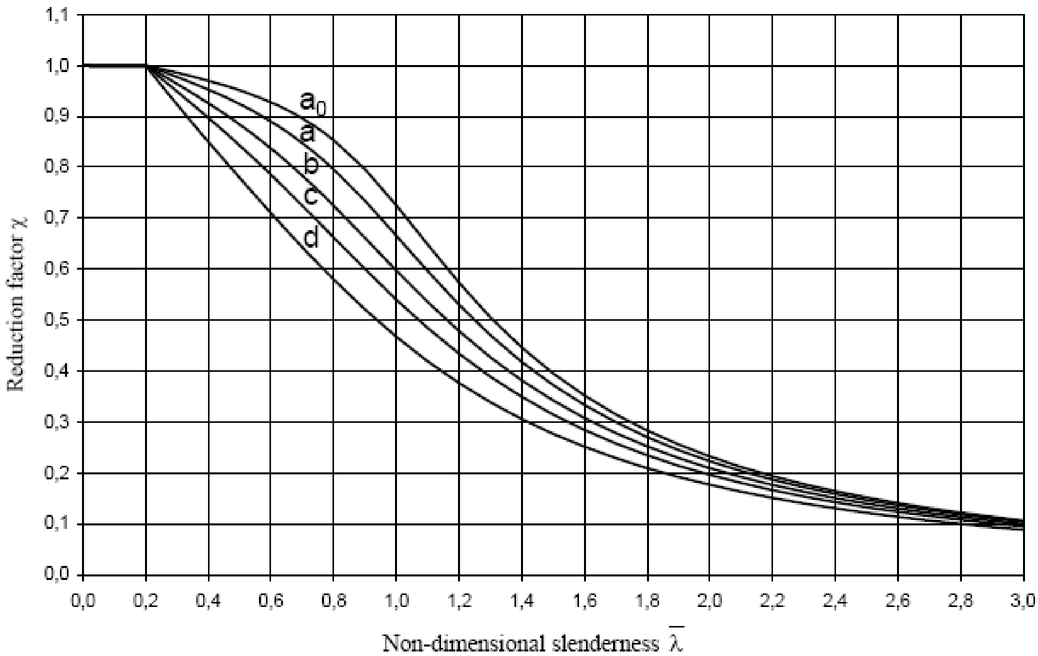


Figure VI-26 Buckling curves (EN 1993-1-1)

Renaud C. (2004) also expressed buckling curves for CFSHS columns subjected to axial load and standard fire exposure. Four fire durations R30, R60, R90 and R120 were considered and five reinforcement ratios were considered, namely 0, 1, 2, 3, 4 and 5%. The curve is divided in two parts: the first part is in the field of low relative slenderness and the second part is in the field of important relative slenderness. The first part is similar to the one defined at ordinary temperature except that the threshold of 0.2 does not appear in the equation and the imperfection parameters are adjusted to change the shape of buckling curve according to the fire duration, cross section dimension and percentage of reinforcement. The second part is another function of relative slenderness and imperfection parameters. According to this reference, the imperfection parameter increases with the fire duration. For the lowest fire duration (R30), it is suggested to adopt $\alpha = 0.21$ (like curve "*a*") for the first part. The second part depends on the fire duration, cross section dimension and the percentage of reinforcement. With the highest percentage of reinforcement (5%), the curve is approximately the same as the curve "*a*" also (Figure VI-27).

With the idea of using the same type of approach as in Eurocode 4 Part 1-1 for composite columns at room temperature but modifying the effective rigidity $(EI)_{md}$ and using curve "*a*" to determine the reduction factor, the load bearing capacity of a column after 10 minutes of fire, $N_{R10,R}$, can be determined as follows:

$$N_{R10,R} = \chi \cdot N_{R10,pl,R}$$

Where χ is the reduction factor depending on the relative slenderness $\bar{\lambda}_{R10}$ which may be defined as: $\bar{\lambda}_{R10} = \sqrt{N_{R10,pl,R} / N_{R10,cr}}$

$N_{R10,pl,R}$ is the plastic resistance of the section after 10 minutes of fire.

$$N_{R10,cr} = \Pi^2 (EI)_{md} / L_{10}^{\ 2}$$

where: L_{10} is the effective length corresponding to a fire resistance of 10 minutes (previously defined)

$(EI)_{md}$ is the modified effective rigidity of the section after 10 minutes of fire

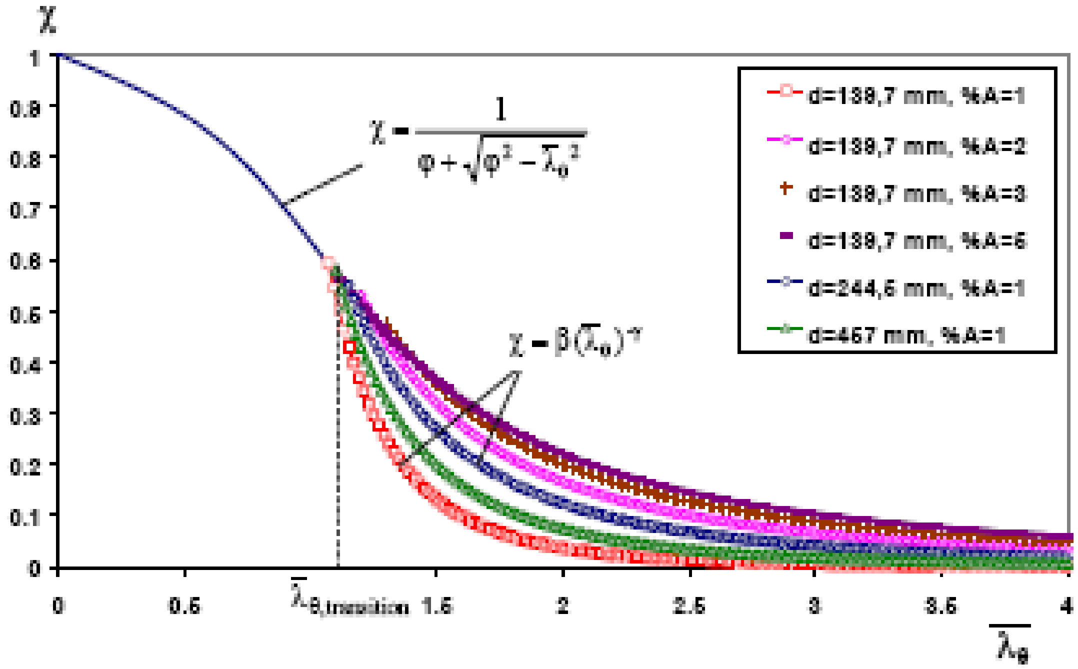


Figure VI-27 (Renaud C. (2004)) - Buckling curves for composite columns subjected to axial load and standard fire exposure (d is the cross section dimension, %A is the reinforcement ratio)

It should be reminded that after 10 minutes of fire exposure of CFSHS columns, the temperatures in reinforcing steel and most of concrete inside are below 100°C, the temperatures in steel wall are approximately 400°C. Let us examine the stress-strain curves of structural steel and concrete at normal temperature and at elevated temperature (Figure VI-28 and Figure VI-29). It is noticeable that at high temperature, the maximum stress level for steel and for concrete does not reach at the same strain value (steel at 400°C reaches the maximum stress at strain of 2% while concrete at 100°C reaches the maximum stress at a strain of 0.4%). Therefore the plastic resistance of the section after 10 minutes of fire is smaller than the perfect plastic normal force calculated with maximum stress levels. Let us consider:

$$N_{R10,pl,R} = \gamma_{R10,a}.A_a \cdot f_y + \gamma_{R10,c}.A_c \cdot f_c + \gamma_{R10,s}.A_s \cdot f_s$$

where $\gamma_{R10,a}$, $\gamma_{R10,c}$, $\gamma_{R10,s}$ are reduction coefficients of steel hollow section, concrete core and steel bars. Strain at failure load of short columns (with various cross-section dimensions, reinforcing bars, concrete grades) after 10 minutes of fire is observed. The mechanical strains at failure of the columns vary from 0.5% to 0.7%. From Figure VI-28 and Figure VI-29, it can be seen that the stresses of steel corresponding to that strain at 400°C and of concrete at temperatures from 20°C to 200°C are about 70% to 90% of the material strength at room temperatures. Using regression analysis with the results from numerous numerical simulations, the proposed values for the reduction coefficients are:

$\gamma_{R10,a} = 0.70$ $\gamma_{R10,c} = 0.9$ $\gamma_{R10,s} = 1.0$

A similar procedure is used for determining the modified effective rigidity of the section after 10 minutes of fire $(EI)_{md}$. Let us consider: $(EI)_{md} = \varphi_{a,R10}.E_a.I_a + \varphi_{c,R10}.E_c.I_c + \varphi_{s,R10}.E_s.I_s$

where $\varphi_{a,R10}$, $\varphi_{c,R10}$, $\varphi_{s,R10}$ are reduction coefficients of steel hollow section, concrete core and steel bar depending on the effect of thermal stresses after 10 minutes in fire of the column.

Numerous numerical simulations have been performed with various cross-section dimensions, concrete grades, concrete covers and steel bars. The following values are chosen to obtain a satisfactory correlation between numerical results by SAFIR code and results calculated with $(EI)_{md}$,using curve "*a*" of European buckling curves:

$\varphi_{a,R10} = 0.20$ $\varphi_{c,R10} = 0.15$ $\varphi_{s,R10} = 0.90$

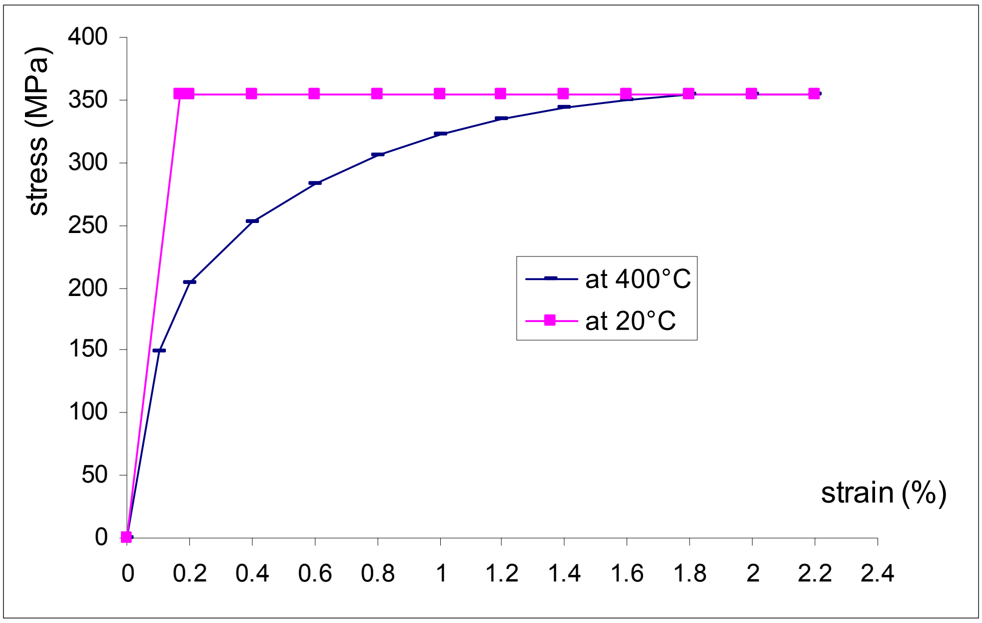


Figure VI-28 Mathematical model for stress-strain relationship of structural steel (according to EN 1994-1-2)

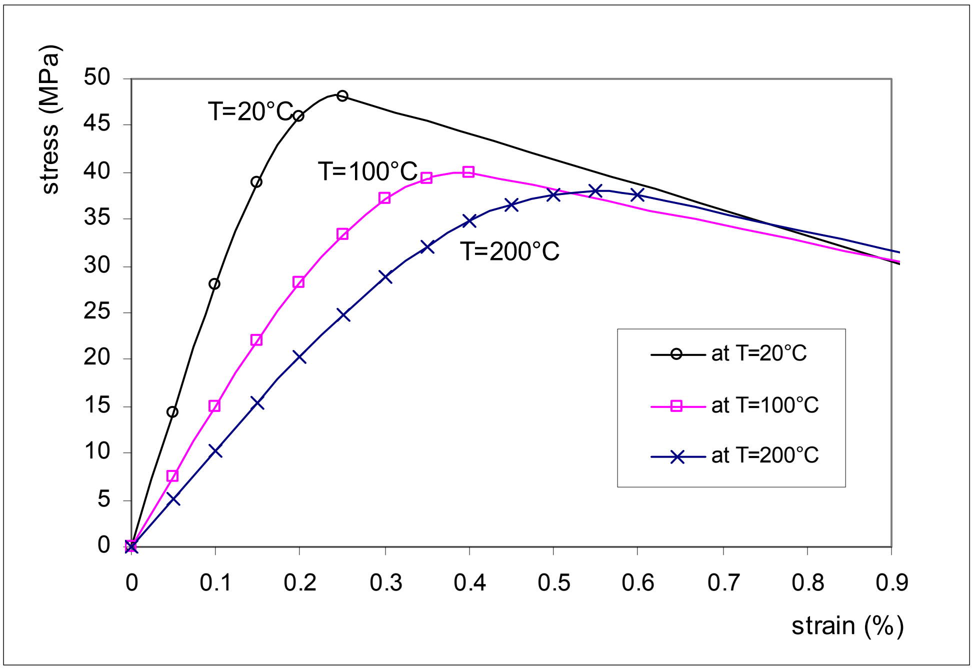


Figure VI-29 Mathematical model for stress-strain relationship of concrete under compression (according to EN 1994-1-2)

The small values of $\varphi_{a,R10}$ and $\varphi_{c,R10}$ can be explained as follows:

- Although the elastic modulus of steel at about 400°C still keeps 70% of the elastic modulus at room temperature, the reduction coefficient of steel hollow section is small ($\varphi_{a,R10} = 0.20$) because at failure, the tangent modulus of steel is much smaller than the elastic modulus (corresponding to the strain from 0.2% to 0.3%) (Figure VI-28).

- At failure, the tangent modulus of many concrete fibers is very small, some are on the descending branch of the stress-strain curve, and some have cracked in tension (in slender columns failure occurs by buckling).

There is a good agreement between numerical results by SAFIR code and results calculated with $(EI)_{md}$, using curve "*a*" of European buckling curves (Figure VI-30 to Figure VI-35). This demonstrates that it is acceptable to use the simplified method suggested in Eurocode 1994-1-1 to calculate the ultimate load of a column after 10 minutes of fire with modified effective rigidity of the section and using buckling curve "*a*".

In the reference Renaud C. (2004), the load bearing capacity of a composite column is also determined from an appropriate buckling column curve which relates load capacity to the plastic load and the elastic critical load using the same approach as the one for composite columns at room temperature. The fire durations R30, R60, R90 and R120 have been considered. The design value of the effective rigidity of the cross section is calculated using the reduction factor coefficients showed in tables. Although these tables are not built for a fire duration of 10 minutes, extrapolations from these tables seem to agree with the reduction coefficients proposed in this research.

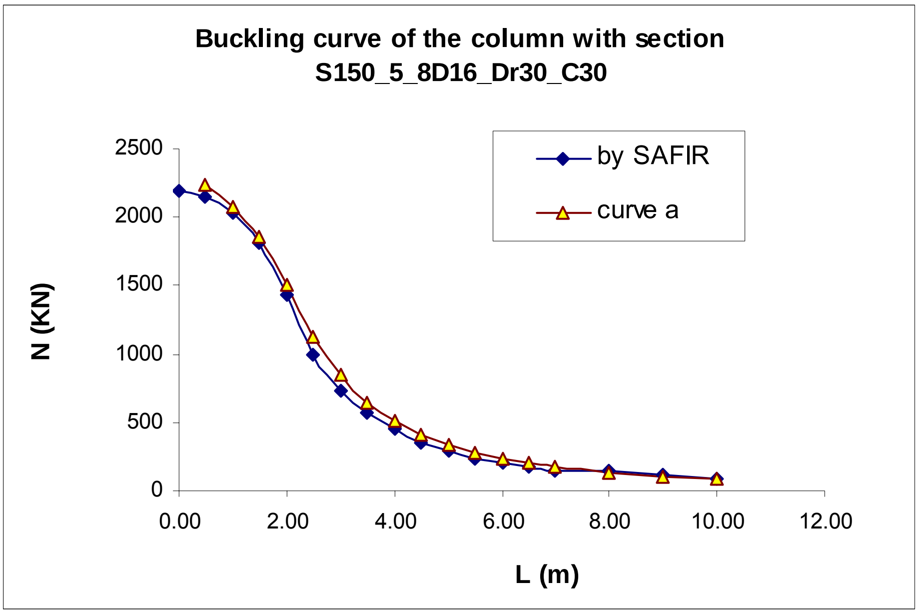


Figure VI-30 Correlation between numerical results and results calculated by the proposed simplified method for the column with section S150-5- C30

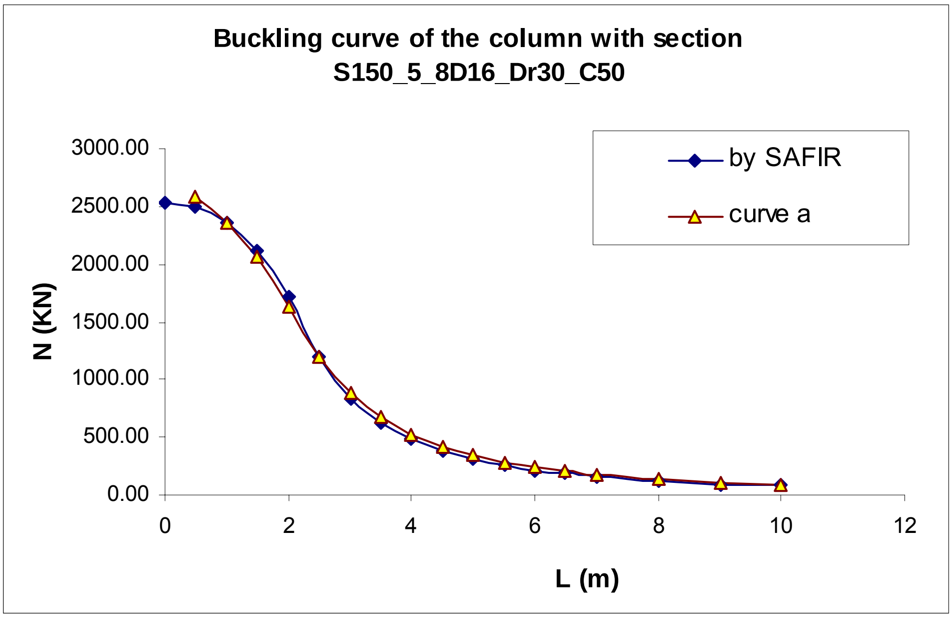


Figure VI-31 Correlation between numerical results and results calculated by the proposed simplified method for the column with section S150-5- C50

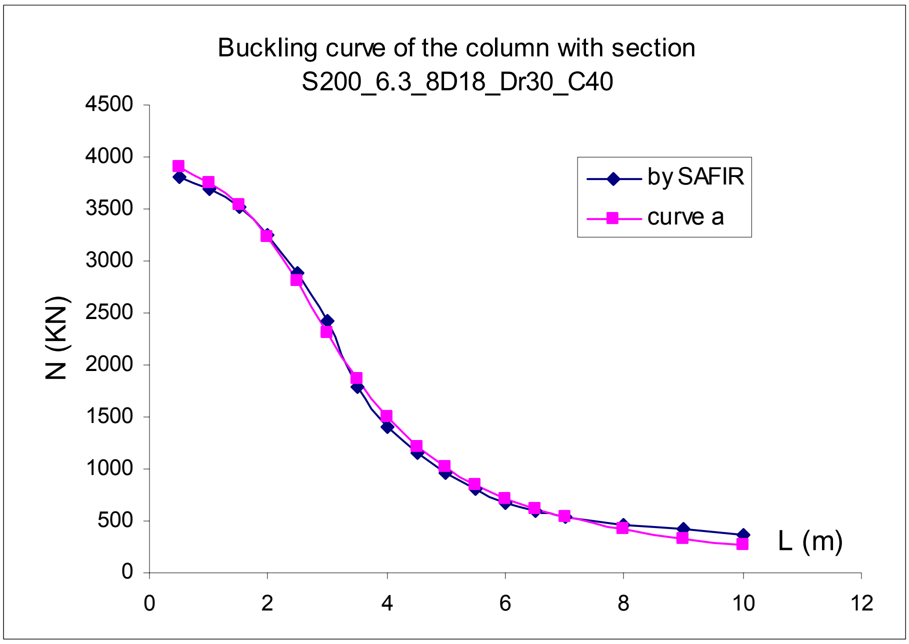


Figure VI-32 Correlation between numerical results and results calculated by the proposed simplified method for the column with section S200-6.3- 8D18

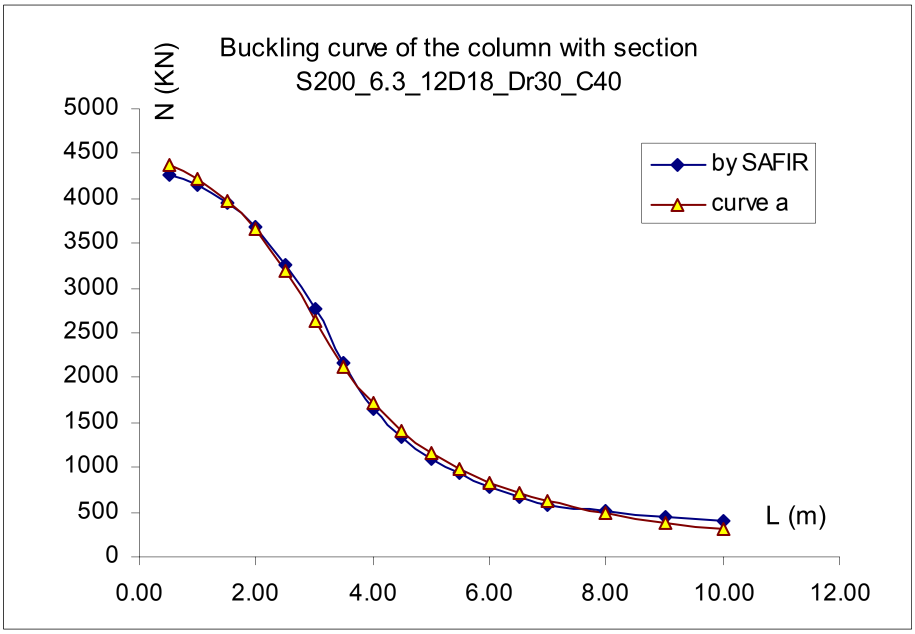


Figure VI-33 Correlation between numerical results and results calculated by the proposed simplified method for the column with section S200-6.3-12D18

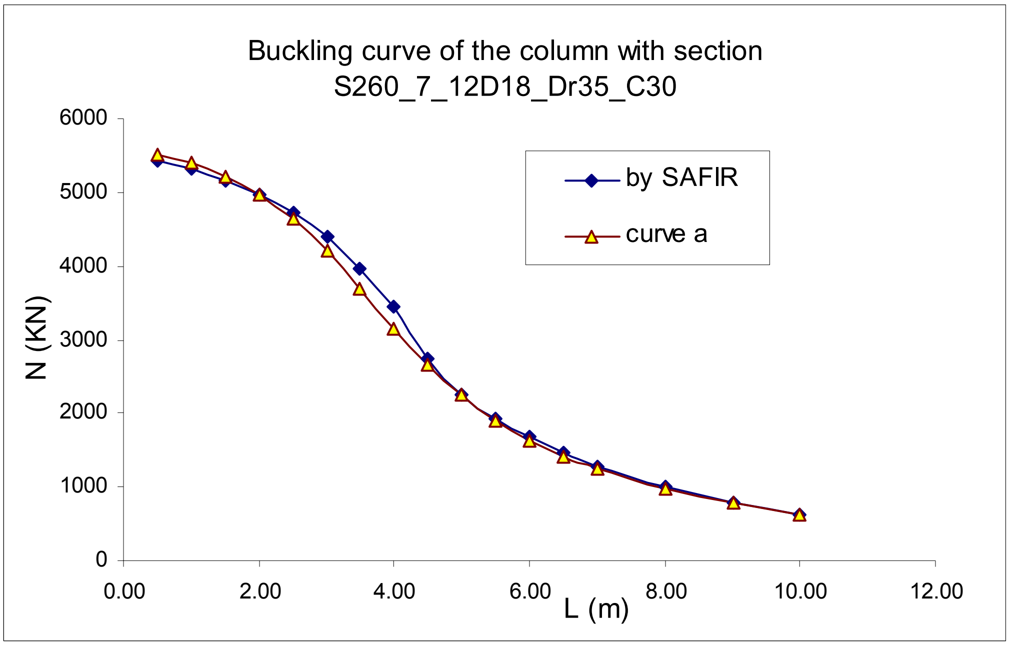


Figure VI-34 Correlation between numerical results and results calculated by the proposed simplified method for the column with section S260-7

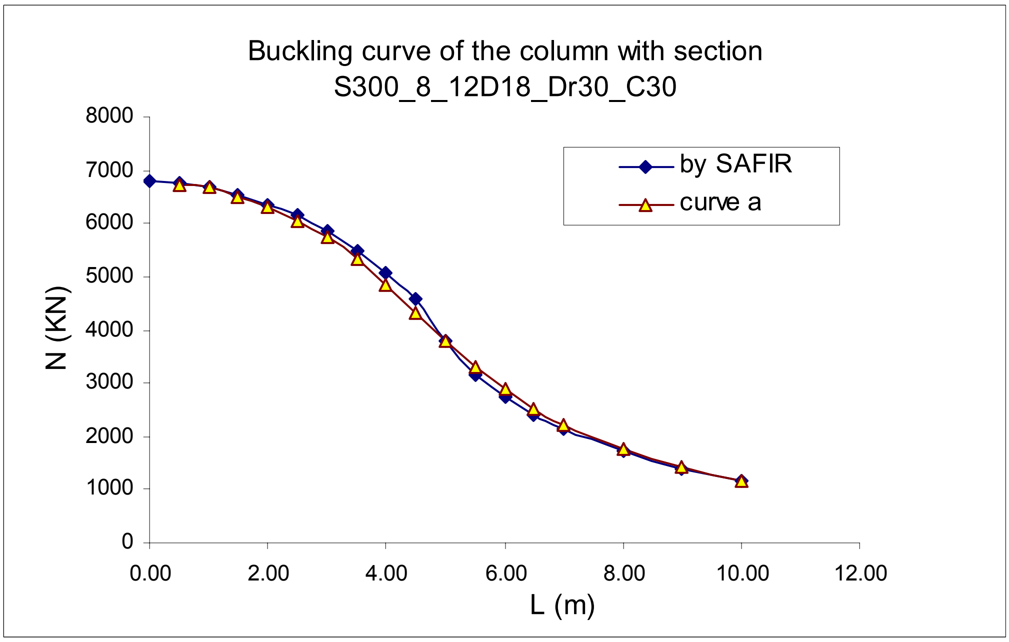


Figure VI-35 Correlation between numerical results and results calculated by the proposed simplified method for the column with section S260-9

VI.1.2.3 Procedure for determining the fire resistance of CFSHS column under centrical load

Based on the previous study, the fire resistance of a CFSHS column can be calculated as follows:

1. Determine the buckling curve of the column after 10 minutes of fire using curve "a" and modified effective rigidity $(EI)_{md} = \varphi_{a,R10}.E_a.I_a + \varphi_{c,R10}.E_c.I_c + \varphi_{s,R10}.E_s.I_s$

where $\varphi_{a,R10} = 0.20$ $\varphi_{c,R10} = 0.15$ $\varphi_{s,R10} = 0.90$

The modified plastic resistance of the section is: $N_{R10,pl,R} = \gamma_{R10,a}.A_a f_y + \gamma_{R10,c}.A_c f_c + \gamma_{R10,s}.A_s f_s$

where $\gamma_{R10,a} = 0.90$ $\gamma_{R10,c} = 0.95$ $\gamma_{R10,s} = 1.0$

2. Based on this buckling curve, with the load N_{fi}, the corresponding length L_{10} can be determined

3. The fire resistance of the short column can be calculated from Eq.(VI.1)

4. The fire resistance of the slender column can be calculated from Eq.(VI.3))

VI.2 Fire resistance of columns under eccentric load

The extension of the simple calculation method developed for axially loaded columns to a method for the fire resistance of eccentrically loaded SHS columns filled with concrete is a very complex problem and extensive research studies should be performed on this matter before proposing a simple design tool.

In this chapter we have reproduced simulations made with the computer code SAFIR based on the following considerations:

- First of all, it has been noticed that the fire resistance of CFSHS columns does not depend significantly on the strength of the external steel wall, since the temperature of this wall is very high near failure. Therefore, under fire conditions, the behaviour of the column is close to the one of a reinforced concrete column.
- Investigations has been made with SAFIR keeping the rate of loading constant, the ultimate load at room temperature being calculated without taking into account the external steel wall. Therefore, simulations have been made by keeping $N_{fi} / N_{u.fy=0}$ constant where:

  N_{fi} : eccentrical load applied under fire conditions;

  $N_{u.fy=0}$: ultimate load at room temperature of the column without the external steel wall with the eccentricity considered

This has been investigated for a lot of values of significant parameters affecting the fire resistance such as cross-section dimension (200mm, 260mm and 300mm), concrete strength (30 MPa, 40 MPa and 50 MPa), column length (from 2 m to 7 m), reinforcement ratio (from 3.5% to 10%), and eccentricity of loading (from 0 to 1.5b namely 0.125b, 0.25b, 0.5b, b and 1.5b, where b is the section dimension).

Some examples are shown from Figure VI-36 to Figure VI-43. It can be seen that the eccentricity of loading has a small influence on the fire resistance of columns subjected to the same $N_{fi} / N_{u.fy=0}$ ratio. Observing carefully the changing fire resistance with eccentricity (provided the same $N_{fi} / N_{u.fy=0}$ ratio), it can be seen that the bigger eccentricity the higher fire resistance (Figure VI-44). So it is safe if the fire resistance of eccentrical loaded column is taken equal to the fire resistance of centrical loaded column provided the same $N_{fi} / N_{u.fy=0}$ ratio.

What has been found out here should be confirmed by additional research works before it is accepted as a final conclusion. Particularly for columns with moderate slenderness, the failure mechanism is completely different for an eccentricity of 0.125b (mainly crushing in compression) and for eccentricity of 1.5b (mainly failure in bending).

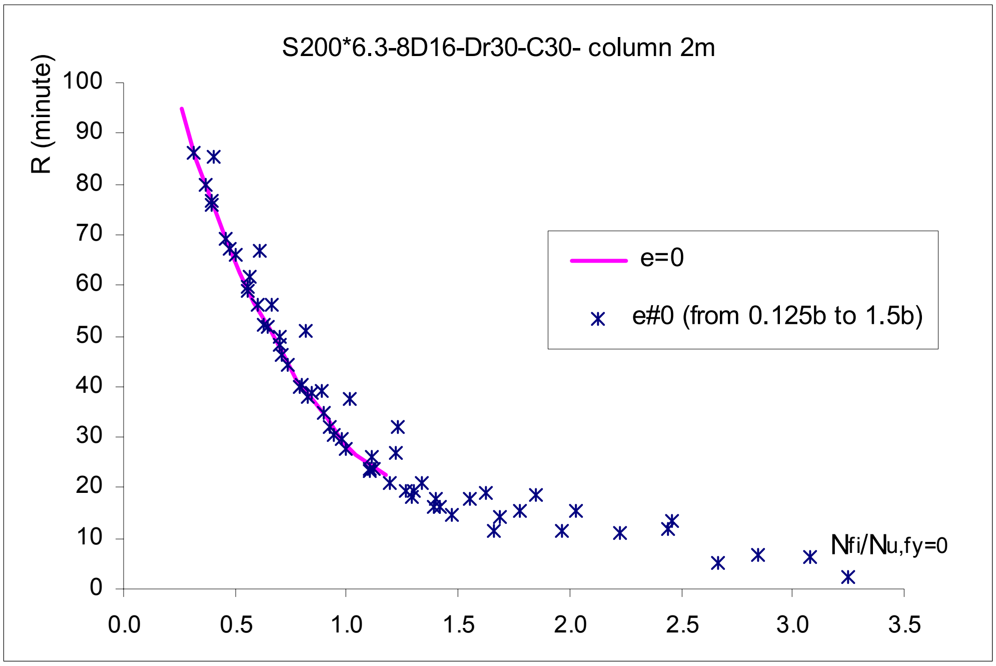


Figure VI-36 Relationship between the fire resistance and $\frac{N_{fi}}{N_{u.fy=0}}$ with various eccentricities of loading for column section S200-6.3, 8ø16 reinforcing bars, column length = 2 m

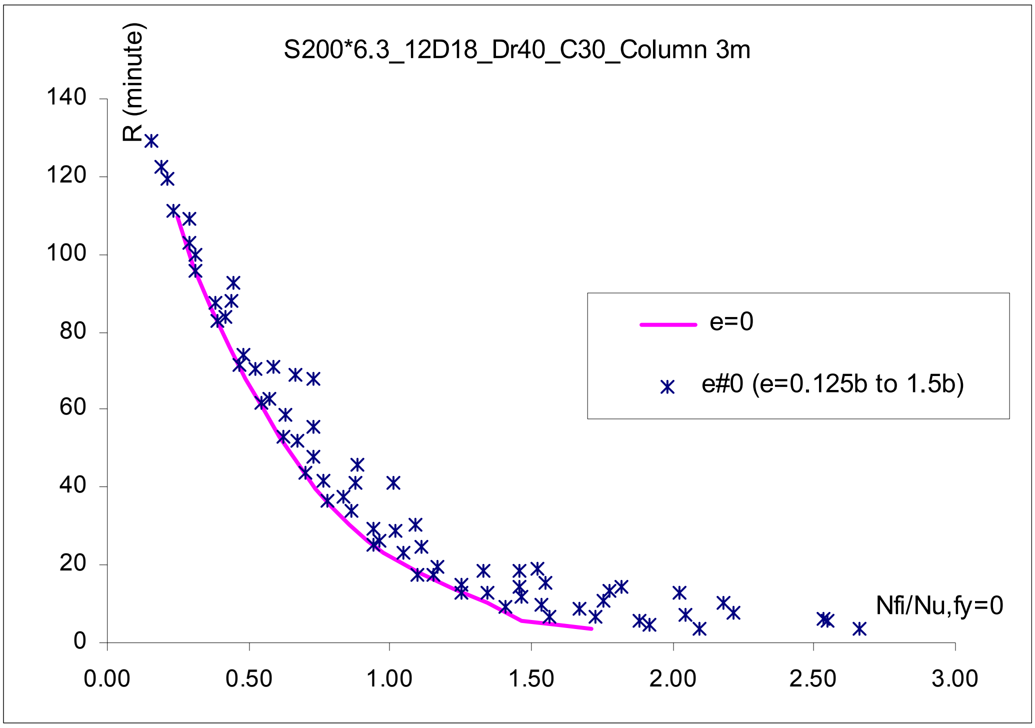


Figure VI-37 Relationship between the fire resistance and $\frac{N_{fi}}{N_{u.fy=0}}$ with various eccentricities of loading for column section S200-6.3, 12ø18 reinforcing bars, column length = 3 m

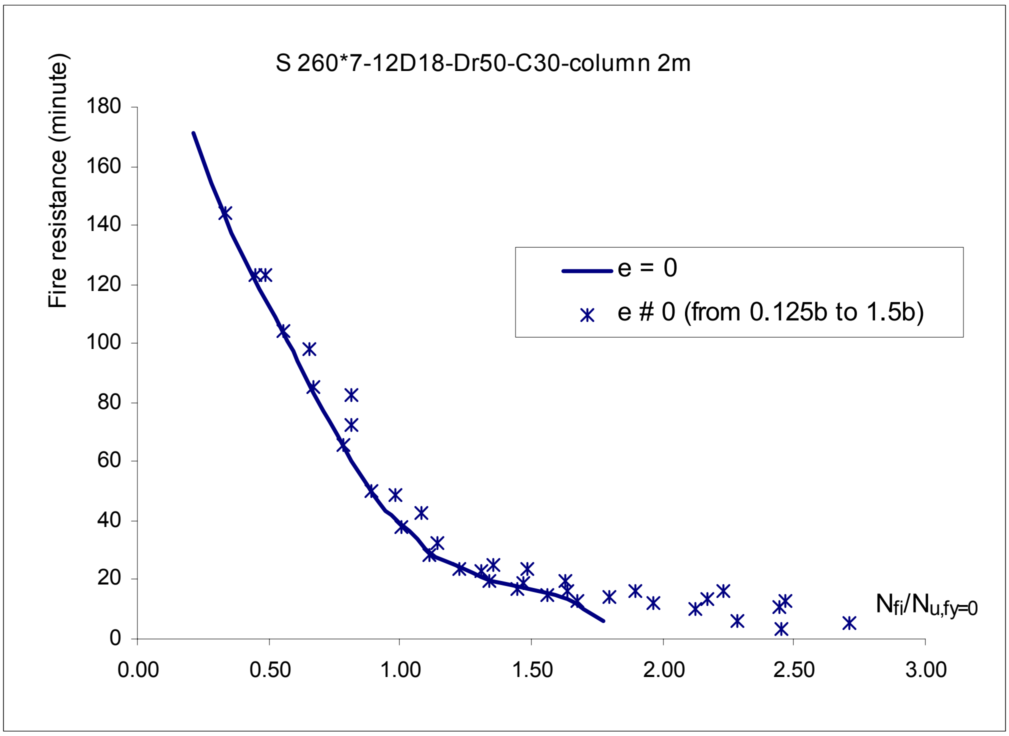


Figure VI-38 Relationship between the fire resistance and $\frac{N_{fi}}{N_{u.fy=0}}$ with various eccentricities of loading for column section S260-7, 16ø18 reinforcing bars, column length = 3 m

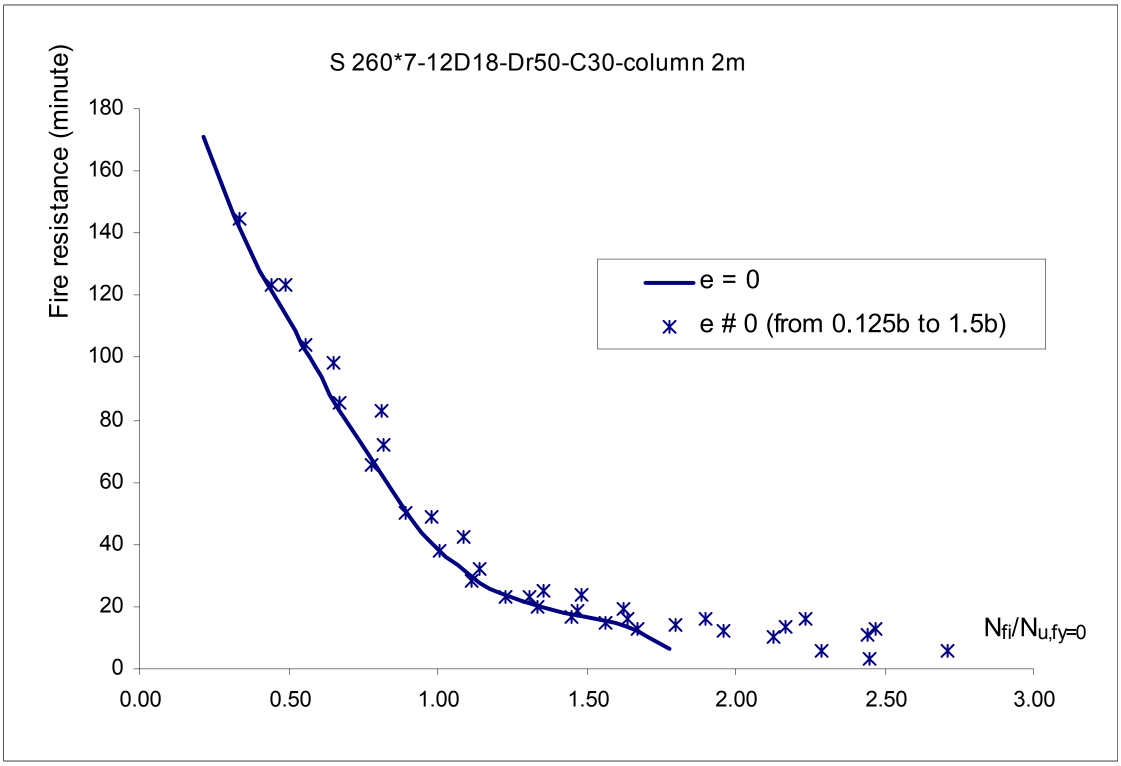


Figure VI-39 Relationship between the fire resistance and $\frac{N_{fi}}{N_{u.fy=0}}$ with various eccentricities of loading for column section S260-7, 12ø18 reinforcing bars, column length = 2 m

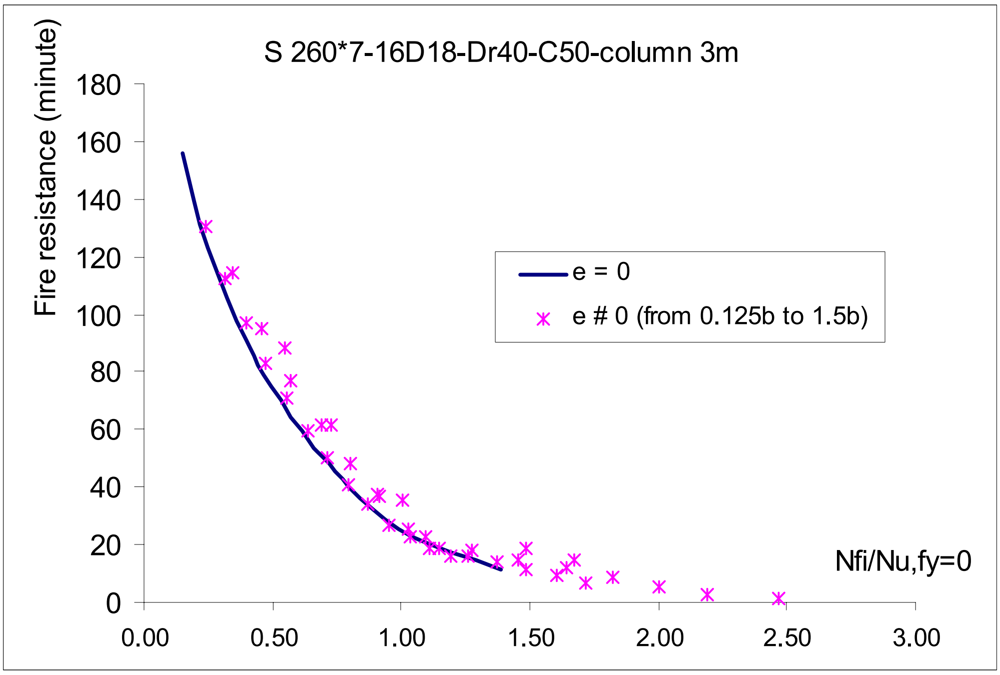


Figure VI-40 Relationship between the fire resistance and $\frac{N_{fi}}{N_{u.fy=0}}$ with various eccentricities of loading for column section S260-7, 16ø18 reinforcing bars, column length = 3 m

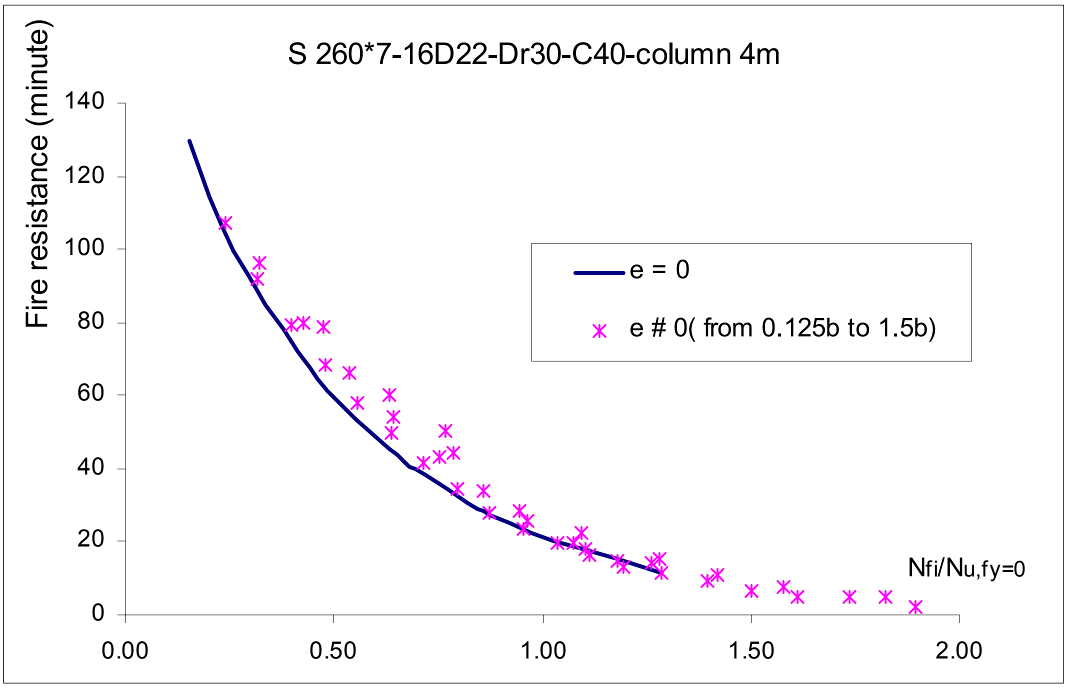


Figure VI-41 Relationship between the fire resistance and $\frac{N_{fi}}{N_{u.fy=0}}$ with various eccentricities of loading for column section S260-7, 16ø22 reinforcing bars, column length = 4 m

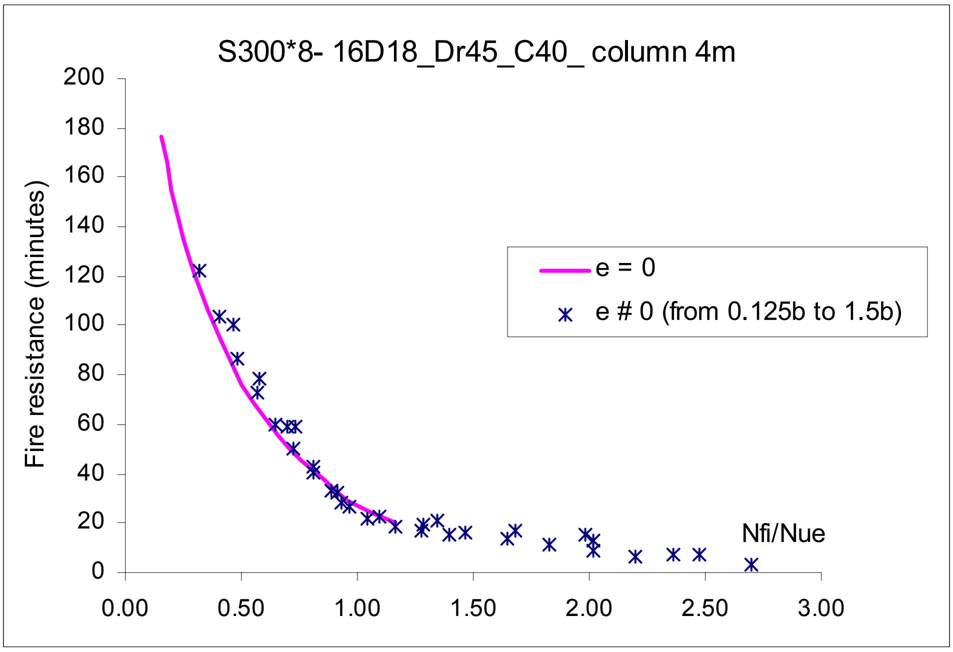


Figure VI-42 Relationship between the fire resistance and $\frac{N_{fi}}{N_{u.fy=0}}$ with various eccentricities of loading for column section S300-8, 16ø18 reinforcing bars, column length = 4 m

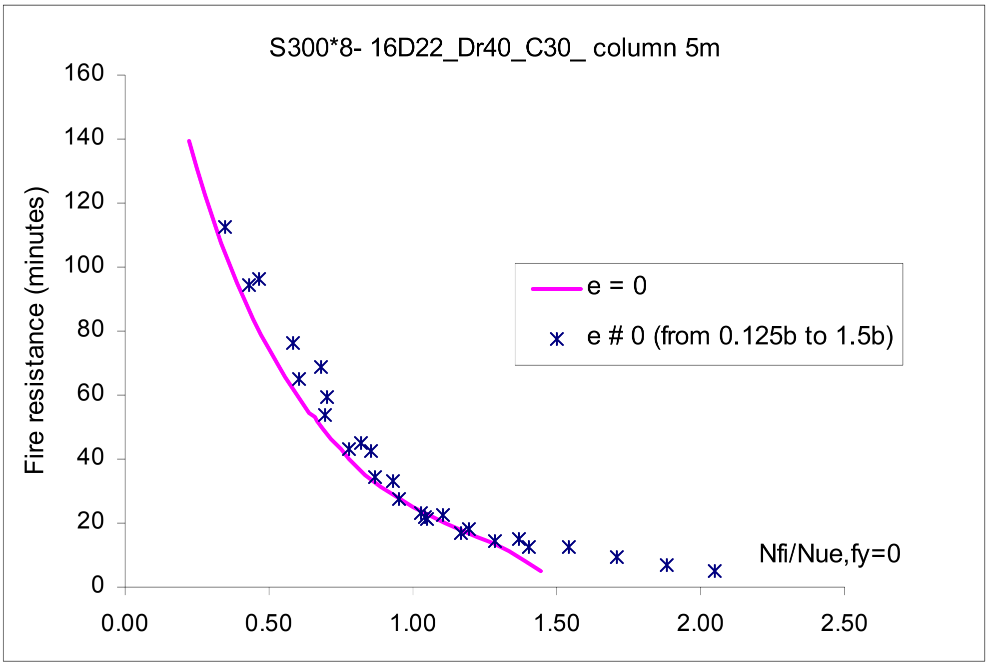


Figure VI-43 Relationship between the fire resistance and $\frac{N_{fi}}{N_{u.fy=0}}$ with various eccentricities of loading for column section S300-8, 16ø22 reinforcing bars, column length = 5 m

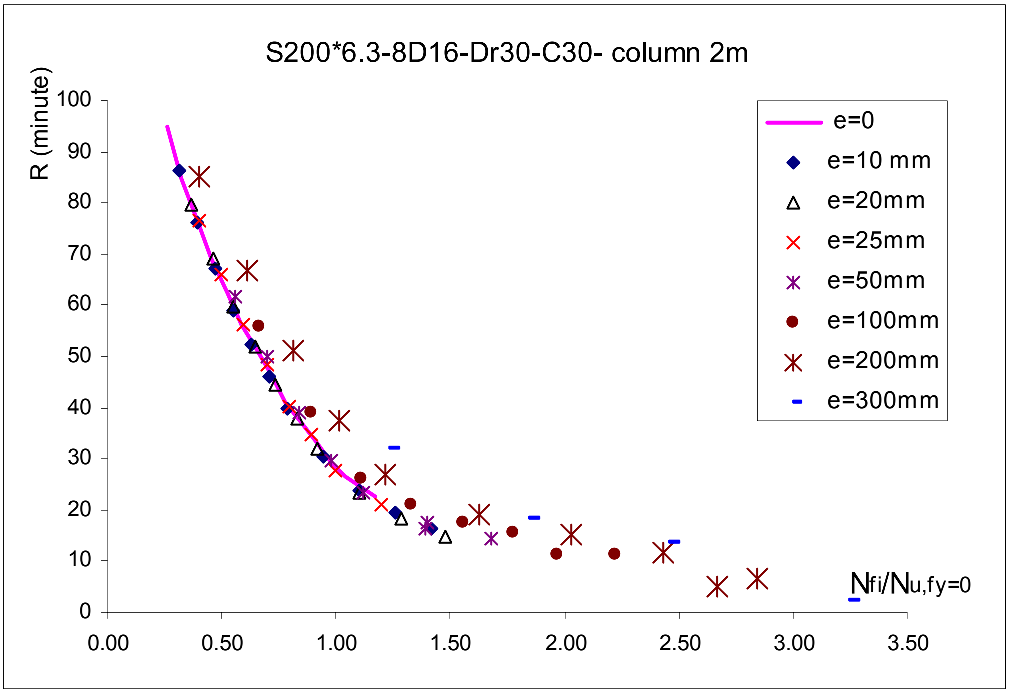


Figure VI-44 Relationship between the fire resistance and $\frac{N_{fi}}{N_{u.fy=0}}$ with various eccentricities of loading for column section S200-6.3, 8ø16 reinforcing bars, column length = 2 m

CHAPTER VII: NUMERICAL CALCULATIONS OF THE FIRE RESISTANCE OF LARGER PROFILES AND PRACTICAL RECOMMENDATIONS

VII.1 Numerical calculations

Chapter III, IV and V concentrated on CFSHS columns with small cross-section dimensions only (less than 300 mm). This is because it was thought that self-compacting concrete is needed primarily in small sections with dense reinforcement or embedded steel profile where it is quite difficult to cast normal concrete. Another reason why small cross-sections were chosen is related to the testing facilities, and mainly the maximum load that could be applied for the tests at ordinary temperatures at ULB and at elevated temperatures at ULg. These maximum loads are respectively 10000 KN and 3000 KN, but it was thought preferable to limit these loads to 6000 KN and 2000 KN. Experimental research described in chapter V indicates that a fire resistance of such small sectional columns is quite low. The columns have the fire resistance of only 22 minutes to 43 minutes when the applied load is equal to 50% of the ultimate load at room temperature (Table V.3). It is however possible to envisage larger section where the use of SCC would be highly desirable. It would be the case when the distance between the external tube and the internal profile is small. It was not possible to test then experimentally, but numerical simulations can be performed. It is hoped that the fire resistance of larger cross-section columns improves significantly, and numerical calculations of the fire resistance of larger profiles have been performed using the computer code SAFIR. These calculations are valid for both SCC and normal concrete infilled because it has been shown in this study that the thermal and mechanical properties of SCC are very close to those of normal vibrated concrete. The main objective of these numerical calculations is to provide data for immediate use by practical engineers.

Sixteen cross-sections have been studied. Six of them are profiles used in the experimental research described in chapter V. The ten other cross-sections are similar to the cross-section types of the profiles used in the experimental research, but larger dimensions. Two values of column length are considered: 3 m and 4 m. For each column the fire resistances is calculated under centrical compressive load, compressive load with an eccentricity of 1 cm and compressive load with an eccentricity of 5 cm. Four values of the load ratio, namely, 0.3, 0.4, 0.5, and 0.6 have been adopted.

The assumptions used in numerical simulations have been described in part III.3.2 and III.3.2.

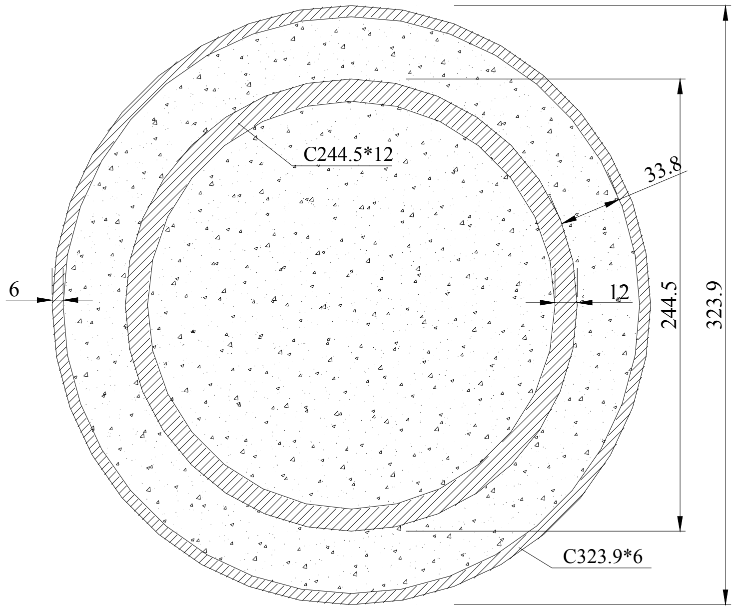
The parameters considered in this chapter are given below:

- The same water content 4% in weight is adopted;
- The quality of steel f_y = 355 MPa is chosen for all steel profiles;
- Two values of column length are considered: 3 m and 4 m. The maximum value of the initial deflection is L/500. It has always been considered that the effect of this eccentricity and the one of loading eccentricity are cumulative.

Firstly, a column was calculated with three values of concrete strength: 30 MPa, 40 MPa and 50 MPa. Results in Table VII.1 and Table VII.2 show that concrete strength affects insignificantly the fire resistance of the column if the load ratio is constant. Therefore, only one value of concrete strength f_c = 40 MPa is considered to limit the number of calculations.

Results from numerical simulations are shown in Appendix 3.

Profile 16



Profile 16 - column 3 m - concrete strength = 30 MPa					
		Area (cm2)	Strength (MPa)		
External steel wall		59.62	355		
Concrete		672.97	30		
Internal steel profile		87.20	355		
Npl,Rk =	7231	KN			
χ = Nu/Npl,Rk =	0.91				
Ultimate load at normal temperature		Load applied in fire		Load ratio	Fire resistance
Load Nu (KN)	Eccentricity e (cm)	Load Nfi (KN)	Eccentricity e (cm)	Nfi/Nu	Rf (minute)
6590	0	1977	0	0.30	117.02
6590	0	2636	0	0.40	81.15
6590	0	3295	0	0.50	50.65
6590	0	3954	0	0.60	35.29
5889	1	1767	1	0.30	119.44
5889	1	2356	1	0.40	84.54
5889	1	2944	1	0.50	51.33
5889	1	3533	1	0.60	34.88
4219	5	1266	5	0.30	121.73
4219	5	1688	5	0.40	88.00
4219	5	2109	5	0.50	53.42
4219	5	2531	5	0.60	34.17

Table VII.1 Characteristics and calculated fire resistances of column- profile 16 with concrete strength 30 MPa

Profile 16 - column 3 m - concrete strength = 40 MPa			
		Area (cm2)	Strength (MPa)
External steel wall		59.62	355
Concrete		672.97	40
Internal steel profile		87.20	355
Npl,Rk =	7904	KN	
χ = Nu/Npl,Rk =	0.91		

Ultimate load at normal temperature		Load applied in fire		Load ratio	Fire resistance
Load	Eccentricity	Load	Eccentricity	Nfi/Nu	Rf (minute)
Nu (KN)	e (cm)	Nfi (KN)	e (cm)		
7176	0	2153	0	0.30	116.25
7176	0	2871	0	0.40	81.58
7176	0	3588	0	0.50	52.27
7176	0	4306	0	0.60	36.44
6405	1	1922	1	0.30	118.13
6405	1	2562	1	0.40	84.19
6405	1	3203	1	0.50	52.65
6405	1	3843	1	0.60	35.96
4568	5	1370	5	0.30	120.13
4568	5	1827	5	0.40	85.77
4568	5	2284	5	0.50	53.88
4568	5	2741	5	0.60	35.10

Profile 16 - column 3 m - concrete strength = 50 MPa			
		Area (cm2)	Strength (MPa)
External steel wall		59.62	355
Concrete		672.97	50
Internal steel profile		87.20	355
Npl,Rk =	8577	KN	
χ = Nu/Npl,Rk =	0.91		

Ultimate load at normal temperature		Load applied in fire		Load ratio	Fire resistance
Load	Eccentricity	Load	Eccentricity	Nfi/Nu	Rf (minute)
Nu (KN)	e (cm)	Nfi (KN)	e (cm)		
7763	0	2329	0	0.30	115.71
7763	0	3105	0	0.40	82.10
7763	0	3881	0	0.50	53.69
7763	0	4658	0	0.60	37.52
6921	1	2076	1	0.30	117.04
6921	1	2769	1	0.40	83.79
6921	1	3461	1	0.50	53.77
6921	1	4153	1	0.60	36.96
4913	5	1474	5	0.30	118.06
4913	5	1965	5	0.40	83.88
4913	5	2456	5	0.50	54.10
4913	5	2948	5	0.60	35.77

Table VII.2 Characteristics and calculated fire resistances of column- profile 16 with concrete strength 40 MPa and 50 MPa

The following symbols are used in the tables:

$N_{pl,Rk} = A_a \cdot f_y + A_c \cdot f_{ck} + A_s \cdot f_s$ the characteristic value of the plastic resistance to compression of the section

χ : reduction factor for flexural buckling $\chi = N_u / N_{pl,Rk}$

N_{fi} :the compressive load applied to the column under fire condition

N_u : the ultimate compressive load of the column at room temperature

VII.2 Conclusions from numerical simulations

The sixteen cross-sections studied can be divided into five groups of cross-section types:

- Group 1 involves profile 1, profile 5, profile 16, and profile 15;
- Group 2 involves profile 2, profile 7, and profile 12;
- Group 3 involves profile 3, profile 8, and profile 11;
- Group 4 involves profile 4, profile 9, and profile 14;
- Group 5 involves profile 6, profile 10, and profile 13.

Group 3 and 4 were calculated in two cases: buckling around minor axis and around major axis.

The following conclusions can be drawn from the results from numerical simulations:

- The eccentricity of the load does not affect much the fire resistance of a column if the load ratio (N_{fi} / N_u) constant. An increase of load eccentricity even increases slightly the fire resistance. It could be thought that an increase in load eccentricity may result in the reduction of the fire resistance, but the applied load on the column will also be reduced because the column has a constant load ratio, thus leading to a longer fire resistance time. This observation has also been recognized in references Han L.H (2001), Han L.H et al. (2003a,c) ;
- Columns with embedded I steel profile (group 3 and 4) display a reduced fire resistance if buckling occurs around the minor axis. Therefore, during construction, if it is impossible to avoid this type of buckling, it is important to reduce as much as possible the incident eccentricity with respect to this minor axis;
- If the load ratio is constant, the cross-section dimensions affect mainly the fire resistance. Columns with sectional dimensions of about 200 mm (Table A3.1, Table A3.2, Table A3.3, Table A3.5) show a fire resistance of about 50 minutes if the value of the load ratio is 0.3 and a fire resistance of about 25 minutes if the value of the load ratio is 0.5. Columns with sectional dimensions of about 350 mm (Table A3.9, Table A3.10, Table A3.19, Table A3.21) show a fire resistance of about 140 minutes if the value of the load ratio is 0.3 and show a fire resistance of about 60 minutes if the value of the load ratio is 0.5.
- The type of cross-section also affects significantly the fire resistance (compare Table A3.9, Table A3.21, and Table A3.14). Columns with an internal void (group 5) show a lower fire resistance compared to columns with the same cross-sectional

dimension;

- The column length affects the fire resistance of the column in the following way. Column 4 m– profile 15 and column 3 m- profile 5 have the same type of cross-section and the same value of reduction factor for flexural buckling at room temperature $\chi = N_u / N_{pl,Rk}$, but results (Table A3.7 and Table A3.21) show that with the same reduction factor χ (i.e. the same slenderness) and load ratio, the fire resistance of the columns with a larger cross-section is higher.

VII.3 Practical recommendations

Using results from numerical simulations, practical recommendations are presented in Table VII.3 to Table VII.7.

The maximum load ratios N_{fi} / N_u are given to get the fire resistance classes R30, R60, R90 and R120. These load ratios are deduced by linear interpolation and extrapolation from the simulation results of columns under centrical load (e = 0) in Appendix 3.

As mentioned in part VII.2, the eccentricity of the load does not affect much the fire resistance of a column if the load ratio (N_{fi} / N_u) is constant. An increase of load eccentricity even increases slightly the fire resistance. Therefore, the maximum load ratios suggested in Table VII.3 to Table VII.7 deduced from results of columns under centrical load can be used safely for column under eccentric loads.

With sections belonging to group 3 and group 4, columns under centrical load will fail by buckling around the minor axis. But under fire condition, columns of this type should be used almost exclusively when construction detailing in such that buckling will occur around the major axis. Therefore the maximum load ratios are deduced from results of columns buckling around the major axis.

Group 1: Concrete filled circular hollow steel tube containing another circular steel tube

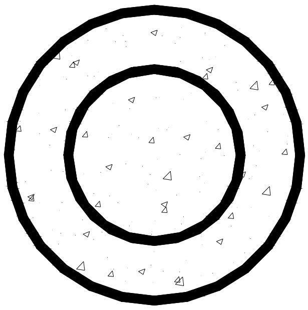


Figure VII.45- Section type of group 1

External steel wall	Internal steel	Fire resistance class	Column length (m)	Maximum load ratio Nfi/Nu
C 219.1 x 5	C 139.7 x 10	R30	3	0.49
		R60	3	0.29
		R90	3	-
		R30	4	0.41
		R60	4	0.25
		R90	4	-
C 273 x 5	C 168.3 x 10	R30	3	0.59
		R60	3	0.39
		R90	3	0.28
		R120	3	-
		R30	4	0.52
		R60	4	0.34
		R90	4	-
C 323.9 x 6	C 244.5 x 12	R30	3	0.64
		R60	3	0.47
		R90	3	0.38
		R120	3	0.29
		R30	4	0.60
		R60	4	0.39
		R90	4	0.30
		R120	4	-
C 355.5 x 6	C 273 x 12	R30	3	0.65
		R60	3	0.52
		R90	3	0.43
		R120	3	0.35
		R30	4	0.63
		R60	4	0.44
		R90	4	0.35
		R120	4	0.27

Table VII.3. Practical table for columns with section type 1

Group 2: Concrete filled circular hollow steel tube containing another square steel tube

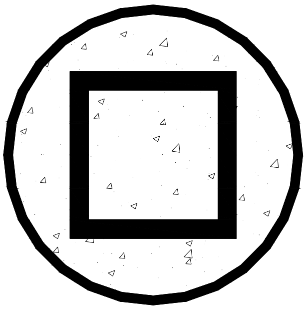


Figure VII.46- Section type of group 2

External steel wall	Internal steel	Fire resistance class	Column length (m)	Maximum load ratio Nfi/Nu
		R30	3	0.49
		R60	3	0.30
C 219.1 x 5	S 120 x 10	R90	3	-
		R30	4	0.42
		R60	4	0.26
		R90	4	-
		R30	3	0.65
		R60	3	0.49
		R90	3	0.38
C 323.9 x 6	S 180 x 12	R120	3	0.31
		R30	4	0.60
		R60	4	0.43
		R90	4	0.36
		R120	4	0.31
		R30	3	0.68
		R60	3	0.54
		R90	3	0.44
C 355.5 x 6	S 200 x 12	R120	3	0.36
		R30	4	0.64
		R60	4	0.48
		R90	4	0.37
		R120	4	0.29

Table VII.4. Practical table for columns with section type 2

Group 3: Concrete filled circular hollow steel tube containing a HEB steel profile

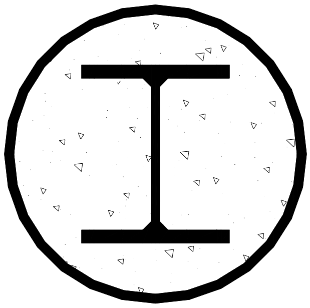


Figure VII.47- Section type of group 3

External steel wall	Internal steel	Fire resistance class	Column length (m)	Maximum load ratio Nfi/Nu
		R30	3	0.47
C 219.1 x 5	HEB 120	R60	3	0.27
		R90	3	-
		R30	4	0.41
		R60	4	0.23
		R90	4	-
		R30	3	0.64
		R60	3	0.48
		R90	3	0.38
C 323.9 x 6	HEB 180	R120	3	0.32
		R30	4	0.59
		R60	4	0.42
		R90	4	0.32
		R30	3	0.65
		R60	3	0.53
		R90	3	0.44
C 355.5 x 6	HEB 200	R120	3	0.37
		R30	4	0.63
		R60	4	0.48
		R90	4	0.37
		R120	4	0.31

Table VII.5. Practical table for columns of section type 3 - buckling around the major axis

Group 4: Concrete filled square hollow steel tube containing a HEB steel profile

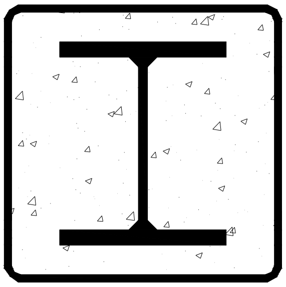


Figure VII.48- Section type of group 4

External steel wall	Internal steel	Fire resistance class	Column length (m)	Maximum load ratio Nfi/Nu
		R30	3	0.47
S 200 x 5	HEB 120	R60	3	0.26
		R90	3	-
		R30	4	0.39
		R60	4	-
		R30	3	0.63
		R60	3	0.48
		R90	3	0.40
S 300 x 6.3	HEB 220	R120	3	0.32
		R30	4	0.60
		R60	4	0.41
		R90	4	0.33
		R30	3	0.65
		R60	3	0.53
		R90	3	0.45
S 350 x 8	HEB 260	R120	3	0.39
		R30	4	0.63
		R60	4	0.47
		R90	4	0.38
		R120	4	0.32

Table VII.6. Practical table for columns of section type 4 - buckling around the major axis

Group 5: Double tubes with an internal void

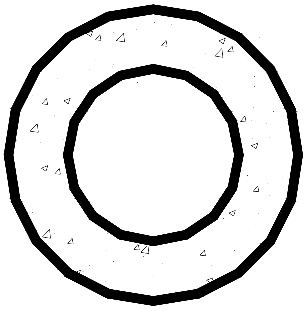


Figure VII.49- Section type of group 5

External steel wall	Internal steel	Fire resistance class	Column length (m)	Maximum load ratio Nfi/Nu
		R30	3	0.57
		R60	3	0.35
C 273 x 5	C 168.3 x 10	R90	3	-
		R30	4	0.52
		R60	4	0.30
		R90	4	-
		R30	3	0.66
		R60	3	0.45
		R90	3	0.33
C 355.5 x 6	C 244.5 x 12	R120	3	-
		R30	4	0.64
		R60	4	0.41
		R90	4	0.28
		R120	4	-
		R30	3	0.68
		R60	3	0.52
		R90	3	0.41
C 406.4 x 6	C 273 x 12	R120	3	0.32
		R30	4	0.67
		R60	4	0.49
		R90	4	0.38
		R120	4	0.34

Table VII.7. Practical table for columns with section type 5

CHAPTER VIII: CONCLUSIONS AND PERSPECTIVES

VIII.1 Conclusions

In this thesis an extensive experimental work and related analytical studies have been carried out in order to investigate the behaviour under ordinary and fire condition of steel hollow section columns filled with self-compacting concrete containing dense reinforcement or another steel profile. The study was undertaken in order to provide information about the behaviour of SHS columns filled with SCC under standard fire tests and then give a simplified method to calculate the fire resistance of this type of column.

Using SAFIR computer code, models for analysis of CFSHS column under ordinary and fire condition have been verified. In thermal analysis, it is confirmed that the temperatures calculated for steel hollow section filled with concrete are in agreement with reality provided a thermal resistance between the steel tube and the concrete core is introduced. The value adopted in this research is $0.013\, m^2K/W$. In structural analysis, the models provide a good estimation of the fire resistance. The axial and lateral deformations of the columns are predicted rather well except that the columns failed in a less ductile manner in simulation. This can be due to the confinement of concrete in the steel tube which is ignored in simulations. Numerical experimentation had to be made in order to get precisely some unknown parameters.

SAFIR program predicted well the temperatures in tested columns filled with SCC by assuming that the properties of SCC are the same as those of normal vibrated concrete, which means that this assumption is correct.

One of the purposes of this work was to give practical tools to consulting engineers. The following procedure has been adopted.

A simplified formula for calculating the fire resistance of short CFSHS columns has been first proposed. The simplified equation predicts the fire resistance of short CFSHS columns in close agreement with the simulation results obtained from advanced numerical models. Then the formula has been extended to slender columns. A procedure is presented for columns with eccentric load.

In this study, the field applicability has been extended: effective length of column from 2 m to 7 m, percentage of reinforcing steel from 3.5 % to 10 %. Sections containing other steel profiles are also considered.

Finally, numerical calculations of the fire resistance of larger profiles (dimensions up to 400 mm) have been performed and additional data are provided for immediate use by practical engineers.

VIII.2 Suggestions for further research

In the framework of a PhD research, the range of study is limited. In this work we have restricted ourselves to small cross-section dimensions, low width (or diameter) to thickness ratio of the steel wall (less than 40) to avoid local buckling, confinement effect of the concrete due to the steel wall neglected. Therefore, a number of possible extensions to this work can be envisaged. These are highlighted below:

- Additional experimental work on thermal and mechanical properties of SCC should be performed. The increase in compressive strength of concrete due to the confinement effect of the concrete core inside the steel wall should be examined and taken into account in the models.
- The range of width (or diameter) to thickness ratio of the steel wall should be extended. Local buckling of the external steel wall should be considered.
- The proposed simple calculation method developed for axially loaded CFSHS columns should be extended to eccentrically loaded CFSHS columns. Although part VI.2 of this thesis presents such a procedure, this matter is very involved and extensive research studies should be performed in order to confirm or invalidate the proposal on the basis of sound physical explanations.