# STEEL HOLLOW COLUMNS FILLED WITH SELF-COMPACTING CONCRETE UNDER FIRE CONDITIONS

**Jean Claude Dotreppe**, Prof., Dept. of Architecture, Geology, Environment and Construction (ArGEnCo), University of Liège, Belgium
**Thi Binh Chu**, Ph.D Research Eng., ArGEnCo, University of Liège, Belgium
**Jean Marc Franssen**, Prof., ArGEnCo, University of Liège, Belgium

## ABSTRACT

*Concrete filled steel hollow section (CFSHS) columns can carry important loads and therefore are used extensively in the construction of high-rise buildings. Steel hollow sections are filled usually with ordinary concrete, but filling problems may arise with small cross sections and dense reinforcement or hollow sections (tubes) surrounding another profile (tube or H section) when the distance between the two profiles is small. For such a configuration, self-compacting concrete can be recommended. Ten columns filled with self-compacting concrete embedding another steel profile have been tested in the Fire Engineering Laboratory of the University of Liege - Belgium. The non linear finite element software SAFIR developed at the University of Liege has been used to simulate the thermal and structural behavior under fire conditions. A good agreement between numerical and experimental results has been obtained. This shows that SAFIR code can predict well the behavior of CFSHS columns and that the properties of self-compacting concrete at high temperatures can be considered to be the same as those of ordinary concrete. Another purpose of this study was to give practical tools to consulting engineers*.

**Keywords**: Columns, Composite Construction, Fire Resistance, Steel Tubes, Self-Compacting Concrete

## INTRODUCTION

Concrete filled steel hollow section (CFSHS) columns are rather expensive, but can carry important loads, and therefore are used extensively in the construction of high-rise buildings. Among the many advantages of this type of profile is the increase of fire resistance with respect to the one of steel tubes alone. They are filled usually with ordinary concrete, but filling problems may arise with small cross sections and dense reinforcement. It will also be the case with hollow sections (tubes) surrounding another profile (tube or H section) when the distance between the two profiles is small. For such a configuration, self-compacting concrete (SCC) can be recommended, provided concrete mix design is made appropriately.

During the last decades experimental and theoretical investigations on fire performance of unprotected steel hollow sections filled with ordinary concrete have been carried out in the world [1,2,3,4]. The calculation and design of this type of elements are now included in codes and standards, like for example in Eurocodes for cold [5] as well as for fire conditions [6]. On the other hand, almost no research study has been performed on this type of profile filled with self-compacting concrete.

In order to analyse the behavior of such columns, it is necessary to investigate the properties of this material at ordinary and elevated temperatures. Research studies performed at ordinary temperatures (see for example reference [7]) indicate that the mechanical properties of SCC do not differ significantly from those of traditional vibrated concrete. There is almost no research work related to elevated temperatures. Reference [8] shows that the variation with temperature of the mechanical properties of SCC appears similar to that of ordinary concrete.

The results presented here are part of a large research program performed at the Universities of Brussels (ordinary temperature conditions) and Liege (fire conditions). This paper is devoted to the behavior under fire conditions.

Ten columns with five different cross-sections have been tested in the Fire Engineering Laboratory of the University of Liege. The dimensions of these sections are rather small due to the limited testing capacities (load applied).

In order to simulate these tests, advanced calculations have been performed using the non linear finite element software SAFIR developed at the University of Liege for the simulation of thermal and structural behavior under ordinary and fire conditions. The aim of these simulations was to analyse the behaviour of these columns and to see whether some calibrations had to be introduced in SAFIR. This numerical experimentation should also indicate whether the properties of SCC at high temperatures can be considered to be the same as those of ordinary concrete.

Another purpose of this study was to give practical tools to consulting engineers in order to perform a quick and safe design of this type of profile under fire conditions.

## EXPERIMENTAL PROGRAM

A total of 10 specimens including eight SCC filled steel hollow section columns without fire protection and two columns protected by intumescent paint have been tested. For each column, temperatures versus fire exposure time at external steel tube, internal steel profile, and concrete core are recorded. The axial and transversal displacements at mid-height of the column are measured. The test parameters were the cross section type and the load ratio (ratio of the load applied in fire condition $N_{fi}$ to the ultimate load of the member at room temperature $N_u$).

## SPECIMEN PREPARATION

All columns are simply supported with the length 3310 mm (including the end plate thickness). There are five different cross sections named profile 1 to profile 5 (Fig.1), each profile being tested twice. The details of each column are listed in Table 1.

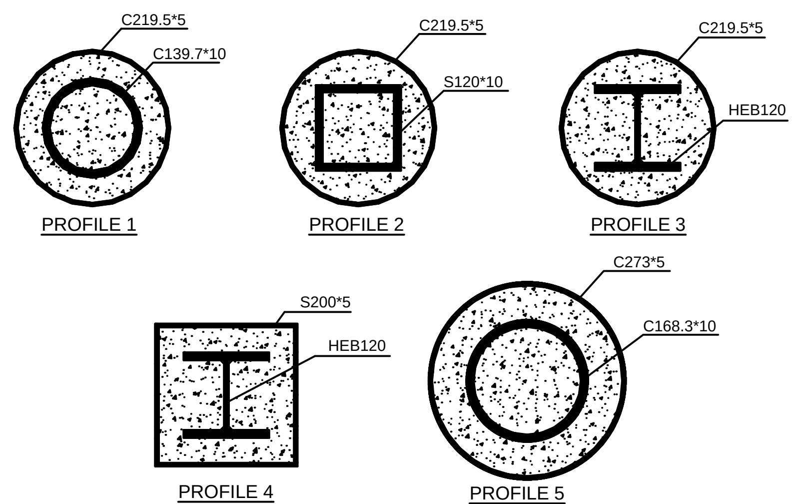


Fig 1 Cross-sections of the columns

Table 1 Structural properties of tested columns

| Test | External steel profile | | Internal steel profile | | Ultimate load at normal temp* | | Test load in fire | | Load ratio |
|---|---|---|---|---|---|---|---|---|---|
| number | Dimensions | Yield strength | Dimensions | Yield strength | Load | Eccentricity | Load | Eccentricity | |
| | (mm) | (MPa) | (mm) | (MPa) | Nu (KN) | e (mm) | Nfi (KN) | e (mm) | Nfi/Nu |
| 1A | 219.1 * 5 | 420 | 139.7 * 10 | 340 | 3000 | 0 | 733 | 0 | 0.24 |
| 1B | 219.1 * 5 | 420 | 139.7 * 10 | 340 | 2253 | 15 | 1126 | 15 | 0.50 |
| 2A | 219.1 * 5 | 420 | 120 * 10 | 349 | 2294 | 15 | 688 | 15 | 0.30 |
| 2B | 219.1 * 5 | 420 | 120 * 10 | 349 | 2489 | 10 | 1244 | 10 | 0.50 |
| 3A | 219.1 * 5 | 420 | HEB 120 | 375 | 2365 | 10 | 946 | 10 | 0.40 |
| 3B (with painting) | 219.1 * 5 | 420 | HEB 120 | 375 | 2241 | 10 | 896 | 10 | 0.40 |
| 4A | 200 * 5 | 510 | HEB 120 | 375 | 2943 | 10 | 1177 | 10 | 0.40 |
| 4B (with painting) | 200 * 5 | 510 | HEB 120 | 375 | 2809 | 10 | 1124 | 10 | 0.40 |
| 5A | 273 * 5 | 420 | 168.3 * 10 | 333 | 3995 | 10 | 1199 | 10 | 0.30 |
| 5B | 273 * 5 | 420 | 168.3 * 10 | 333 | 3995 | 10 | 1998 | 10 | 0.50 |

* The ultimate load at normal temperature has been predicted using SAFIR simulation by assuming that the initial deformation shape of the columns is a semi-sine curve with maximum deflection of L/500.

Columns 1A and 1B, 2A and 2B, and 5A and 5B are identical but the values of the compressive load are different. The cross-section of columns 3A and 3B are identical but column 3A has been tested with the eccentric compressive load creating buckling around the major axis, while column 3B has been tested with buckling around the minor axis. In this case, the fire resistance will be much lower. Therefore, column 3B was protected by intumescent paint. The same is valid for the couple 4A- 4B but the thickness of the painted layer for column 4B is larger.

The compressive strength of self-compacting concrete has been measured on cubes 120 mm at 28 days. The value adopted for the simulation is 35 MPa (corresponding value on cylinders at an age of 3 months).

The mean water content in concrete was determined by oven drying technique. More than 3 months after concreting, the result was approximately 6% in weight. This value has been chosen in simulations.

Eight steam vent holes have been drilled near the ends of each column. Two holes to accommodate the thermocouple cables have been drilled near one face of the tube.

Type K- chromel-alumel thermocouples have been used for measuring temperatures in concrete, internal steel profile and external steel tube at 4 cross sections of the columns.

## TESTING DEVICE

The fire tests have been performed in the Fire Testing Laboratory of the University of Liege-Belgium. The tests have been carried out in accordance with EN 1365-4 [9].

The load is produced by hydraulic jacks with a total capacity of 3000 KN. The jacks are located at the bottom of the furnace chamber. Columns are supported by a fixed beam at the top end and a moving beam at the bottom end. The moving beam is supported by the hydraulic jacks, and thus can move vertically when the jacks produce load.

The vertical furnace is 3250 mm wide and 3250 mm high. At the origin the furnace was built to test only vertical separating elements. Therefore a new part was built and attached to the old part (Fig. 2). The new part has got no burners: all burners are inside the old part of the furnace chamber. The burners can be adjusted to reach easily the desired temperature in the furnace chamber. But thermal gradients are expected between the new part and the old part of the furnace, which will be confirmed by experimental results.

The furnace temperatures are measured by 13 thermoplates which are located about 150 mm from the test specimen, at various heights (Fig. 2). The temperatures measured by the thermoplates are averaged automatically and the average temperature is used as the criterion for controlling the furnace temperature.

In some tests, two strain gauges were attached for measuring the strains at two points situated opposite on the external tube at middle height of the column. These gauges are used to measure the deformation of the column in the loading stage (at room temperature). Using both values, it is possible to predict the curvature of the column, and assuming the deflection to be represented by a sine function, it is also possible to predict the deflection at mid-height of the columns.

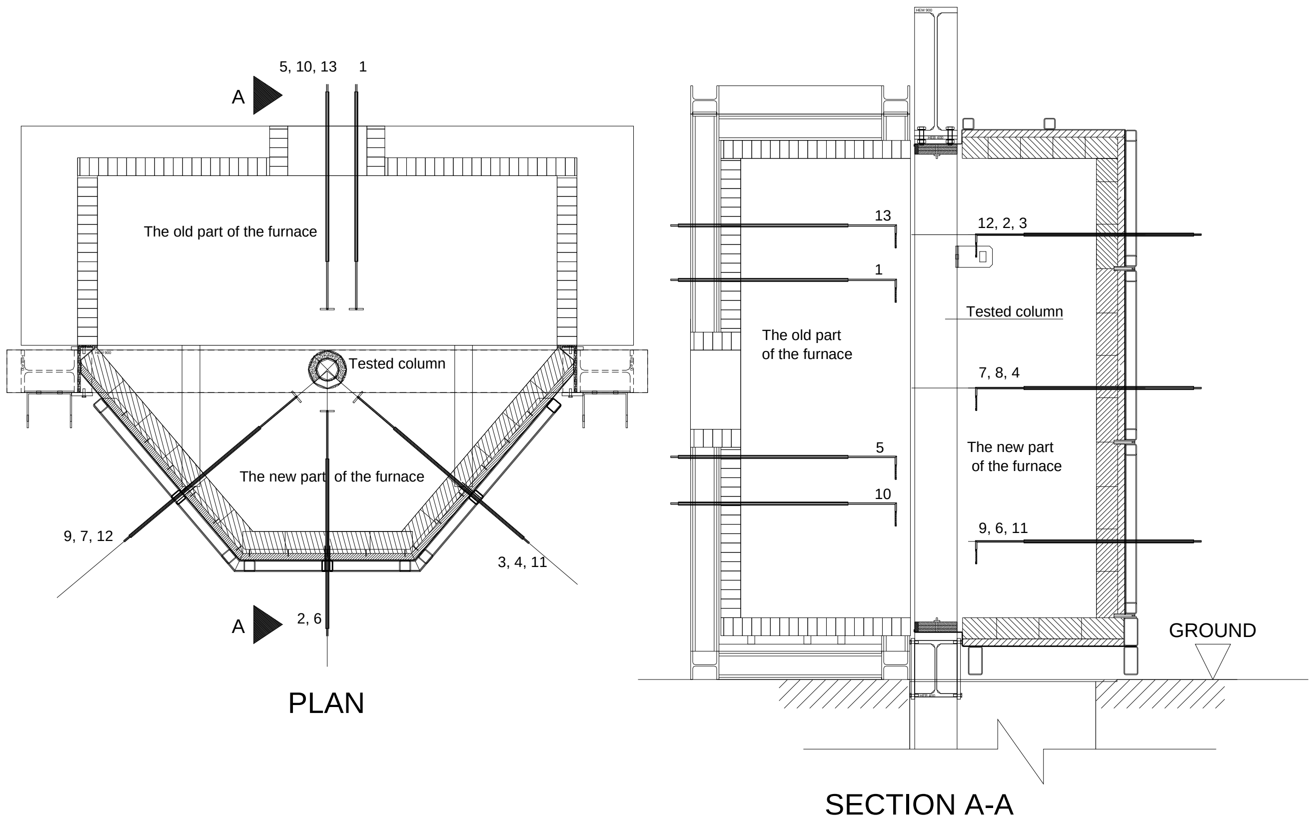


Fig. 2 Location of thermoplates in the furnace

## TEST CONDITIONS AND PROCEDURES

In all tests, the end conditions were hinged. These hinges are made by rolls placed in such a way that lateral displacements of the column have to be perpendicular to the furnace.

The transversal displacement at mid-height of the columns was evaluated using a wire going outside of the furnace. A thermocouple was attached to the wire for measuring the temperature during the fire test and thus calculating the error due to the thermal elongation of the wire.

All columns have been subjected to a load situated between 25% and 50% of the ultimate load at room temperature. All columns except column 1A have been loaded eccentrically. The values of the load and its eccentricity are detailed in Table 1. Loads have been applied at least 15 minutes before the start of the fire test according to the prescriptions of EN 1365-4 [9]. The ISO 834 standard temperature - time curve has been applied in all tests.

The columns were considered to fail when the vertical displacement curve started to go down steeply. At that moment, the test had to be stopped quickly in order to avoid damage in the furnace.

## EXPERIMENTAL RESULTS AND BEHAVIOR OF THE COLUMNS

All columns failed by overall buckling although small marks of local buckling of the steel tube have been observed on columns 1B, 4A, 4B and 5B. A typical failure mode of the tested columns is shown in Fig. 3. Fig. 4 shows local buckling appeared on the steel tube.

The measured fire resistance of all tested columns is shown in Table 2. It can be seen that the increase of the load ratio causes a steep reduction of the fire resistance.

Fig. 3 Column 1B after the test

Fig. 4 Local buckling during test 4A

Table 2 Comparison between calculated and measured fire resistance of tested columns

| Test number | Test load in fire | | Load ratio | Tested fire resistance | Calculated fire resistance | $R_{t\_cal}/R_{t\_test}$ |
|---|---|---|---|---|---|---|
| | Load $N_{fi}$ (KN) | Eccentricity e (mm) | $N_{fi}/N_u$ | $R_{t\text{-}test}$ (minute) | $R_{t\_cal}$ (minute) | |
| Profile 1A | 733 | 0 | 0.24 | 86 | 82 | 0.95 |
| Profile 1B | 1126 | 15 | 0.50 | 22 | 25 | 1.12 |
| Profile 2A | 688 | 15 | 0.30 | 65 | 67 | 1.03 |
| Profile 2B | 1244 | 10 | 0.50 | 43 | 41 | 0.94 |
| Profile 3A | 946 | 10 | 0.40 | 56 | 51 | 0.92 |
| Profile 3B (with painting) | 896 | 10 | 0.40 | 64 | 63 | 0.99 |
| Profile 4A | 1177 | 10 | 0.40 | 39 | 40 | 1.04 |
| Profile 4B (with painting) | 1124 | 10 | 0.40 | 79 | 73 | 0.92 |
| Profile 5A | 1199 | 10 | 0.30 | 104 | 87 | 0.84 |
| Profile 5B | 1998 | 10 | 0.50 | 35 | 41 | 1.16 |
| | | | | | Mean = | 0.99 |
| | | | | | Standard deviation = | 0.10 |

Two columns 3B and 4B are fire protected by intumescent paint. The intumescent painted layer starts swelling at relatively low temperatures (about 200°C to 300°C). The expanded painted layer acts as a thermal barrier that effectively protects the column against rapid increase of temperature. According to the observations made during the test, there were many cracks in the painted layer and they were progressing with temperature (Fig. 5). In test 4B, some parts of the painted layer fell off after about 80 minutes of fire when the temperature at the external surface of the painted layer was close to 1000°C (Fig. 6).

Fig. 5 Intumescent paint of column 4A after the test

Fig. 6 Column 4B after the test

It has been observed that the tested specimens behaved in a relatively ductile manner. At the early stage of heating the steel tube starts to expand and compressive stresses in the concrete core decrease. Then, at high temperatures, the steel loses its strength and stiffness and may locally buckle. Loads are progressively transferred onto concrete core. In the final limit stage, the steel carries almost not load, but keeps confining the concrete core , until the concrete core fails. This behavior was also mentioned in references [1,2] for steel hollow columns filled with plain or reinforced concrete. Columns filled with plain concrete fail in a relatively brittle manner. In this research study experimental results show (Fig. 7) that the structural behavior is rather ductile, which can be explained by the influence of the internal steel profile embedded in concrete.

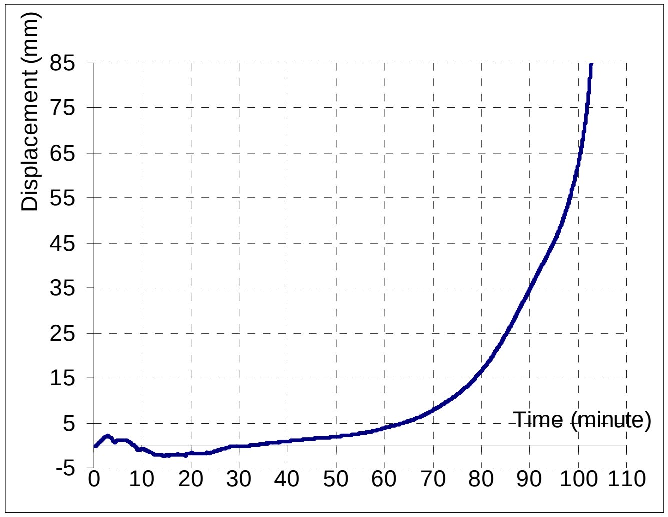


Fig. 7 Transversal displacement of column 5A

## SIMULATIONS AND EVALUATION

All tested columns have been simulated using SAFIR computer code, a non linear finite element software developed at the University of Liege for the simulation of thermal and structural behavior under ordinary and fire conditions [10]. The analysis of a structure exposed to fire consists of two steps. The first step involves predicting the temperature distribution inside the structural members, referred to as "thermal analysis". The second step, named "structural analysis", is carried out in order to determine the mechanical response of the structure due to the thermal effects, since the load is constant during the fire.

### THERMAL ANALYSIS

Assuming the temperatures uniform over the height of the column, the temperatures at the external steel tube, the internal steel profile and the concrete centre point are calculated and compared to the measured values.

The thermal interaction (gap) between external steel section and concrete core is taken into account by means of a fictitious thermal resistance assumed constant along the steel-concrete interface and independent of the temperature. This thermal resistance (R) has been obtained by numerical experimentation, comparing a large number of experimental results with the simulations made by SAFIR. The value adopted is 0.013 $m^2K/W$.

As previously mentioned, thermal gradients are expected between the old part of the furnace with burners and the new part without burner. The average temperatures in each part are calculated and compared in Fig. 8. The differences are less important than expected (about 30°C to 50°C) but they have been introduced in the simulations: the fire boundary of the cross section is divided in two parts.

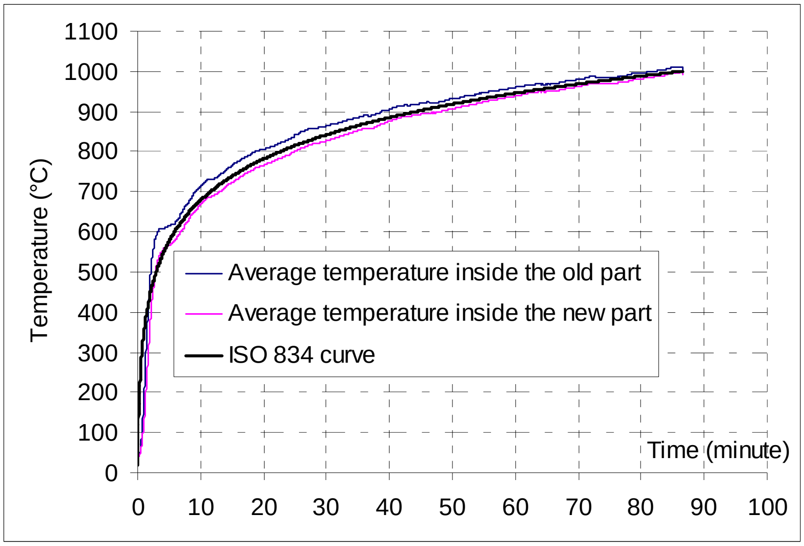


Fig. 8 Temperatures in the furnace of test 1A

Two columns 3B and 4B are fire protected by intumescent paint which swells at high temperature. It is of course impossible to measure the change of thickness of the layer during fire. Therefore, in simulations, it is assumed that the intumescent paint thickness remains unchanged with temperature (fixed at 1mm in calculations), while the characteristic value of the thermal conductivity varies with temperature. The modified thermal conductivity of the painted layer was adjusted to get a good agreement between measured and simulated temperatures in the profile. According to the observations made during the test, there were many cracks in the painted layer and they were progressing with temperature. Therefore the modified thermal conductivity of the painted layer increases for temperatures higher than 700°C as shown in Fig. 9 and Fig. 10. Although the thickness of the painted layers in column 3B and column 4B are different, the global thermal conductivity of the two expanded layers does not differ much.

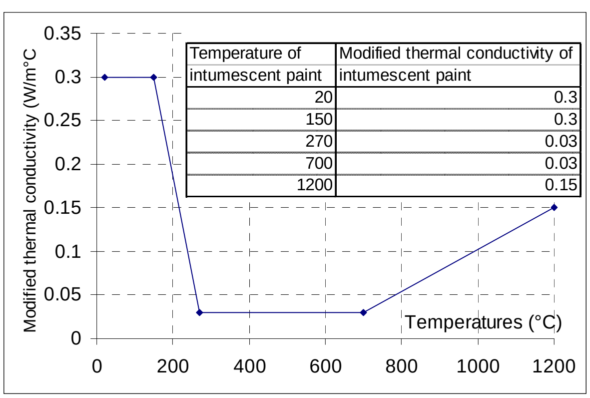


Fig. 9 Modified thermal conductivity of the painted layer of column 3B

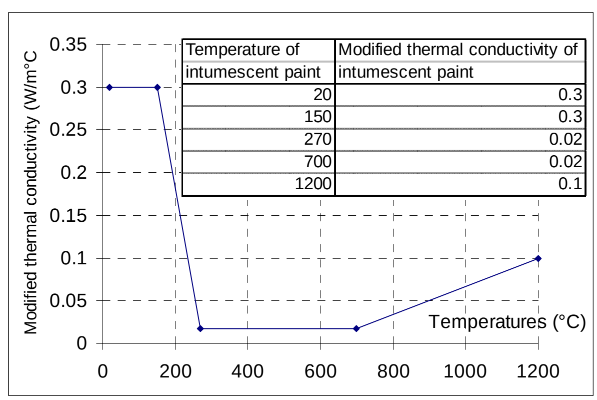


Fig. 10 Modified thermal conductivity of the painted layer of column 4B

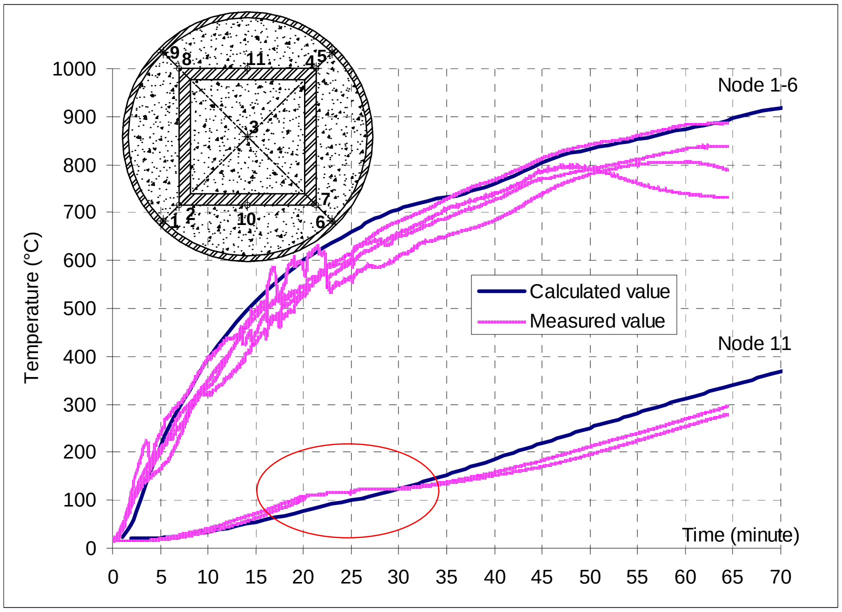


Fig. 11 Temperatures in column 2A

Comparing the calculated and measured temperatures, the following comments can be drawn:

- Calculated temperatures on the external hollow section are in good agreement with the measured temperatures;
- There are systematic differences between calculated and measured temperatures at the internal steel profile: the measured vaporisation stage is longer than the simulated one. Before the evaporation stage, the calculated temperatures at the internal steel profile are lower than the measured values (see Fig. 11). This phenomenon was also mentioned in reference[11]. These differences can be explained by the migration of vapour in concrete (not taken into account in the model. In fact, part of this water moves to the external steel section, but can only escape through eight small holes drilled at the top and at the bottom of the steel tube. Another part of the vapour migrates toward the coldest zones where it condenses again, which results in a slowing down of the vaporisation phase. When this hot vapour reaches the internal steel profile it tends to increase the temperature of the steel. But these differences do not affect much the mechanical properties of materials because there is almost no decrease of the mechanical properties of steel and concrete at about 100°C to 150°C.

## STRUCTURAL ANALYSIS

All columns have been simulated using SAFIR program. The calculated and tested values of the fire resistance are compared in Table 2.

The calculated displacements are compared to the measured ones for all tested columns. A particular result is shown in Fig. 12 and Fig. 13.

At the beginning of the fire test, there is a difference between the measured and the calculated displacements. This has been explained by the particularity of the testing device (moving beam at the bottom end supported by hydraulic jacks). To easily compare vertical displacements, a translation of the measured value has been made in order to get the same value at time 0 for the calculated curve and the moved measured curve.

Comparing the calculated displacements and measured ones the following comments can be made:

- The calculated elongation of the columns is higher than the measured value in the first 5 minutes of testing. After this early stage, the calculated vertical displacements agree well with the measured values;
- The transversal displacement of the columns is sensitive to the assumed initial deformation;
- In most tests, after 2 to 4 minutes of fire, the column deforms toward the old part of the furnace and after some time toward the new part (see Fig. 13). This phenomenon does not appear in simulations if a uniform temperature around the column is adopted (Fig. 14). If the thermal gradient measured in the furnace is introduced, the calculated transversal displacement curve has the same form as the recorded one (Fig. 13). Therefore, the differences in the transversal displacement curves between the calculated and measured results can be explained by the discrepancy between the real temperature field and the one chosen in the model. However the fire resistance is not affected significantly by the thermal gradient in the furnace as has been shown by the simulations. Therefore a uniform temperature around the column can be assumed in the simulations to predict the fire resistance;
- The tested columns behaved in a relatively ductile manner. But in simulations, the columns failed in a less ductile manner: the transversal displacements change steeply from small to large values. These differences need to be explained. Firstly, it has been assumed that the strain hardening (for temperatures below 400°C) of the internal steel tube (not taken into account in the simulations) causes the ductility. A new material model according to Eurocodes [12] introducing the strain hardening was added to SAFIR program. New simulations of the tested columns were made. But the results show that all columns failed when the strains in steel are less than the strain value at the starting point of hardening stage (2%). Therefore, it has been concluded that the strain hardening of carbon steel does not affect the simulations;
- This ductile behaviour could be due to the confinement of the concrete core due to the steel tube. This assumption seems reasonable especially for columns with double tubes because mechanical properties of the internal steel tube do not change much during the fire test, and thus it can confine the lateral expansion of concrete inside. The study of this parameter in CFSHS columns like those examined here could be a perspective for additional research works;
- Local buckling of the external steel was observed in some columns, mainly for square tubes. This phenomenon is ignored in simulations. The question can then be raised whether this parameter may affect significantly the global behaviour of the column. Analysing the stresses in external steel, it is seen that after only about 30 minutes in fire, the external steel looses almost all its strength and stiffness, and the load is transferred to

the concrete core. Therefore, it is believed that local buckling of the external steel tube affects little the fire resistance duration of the columns.

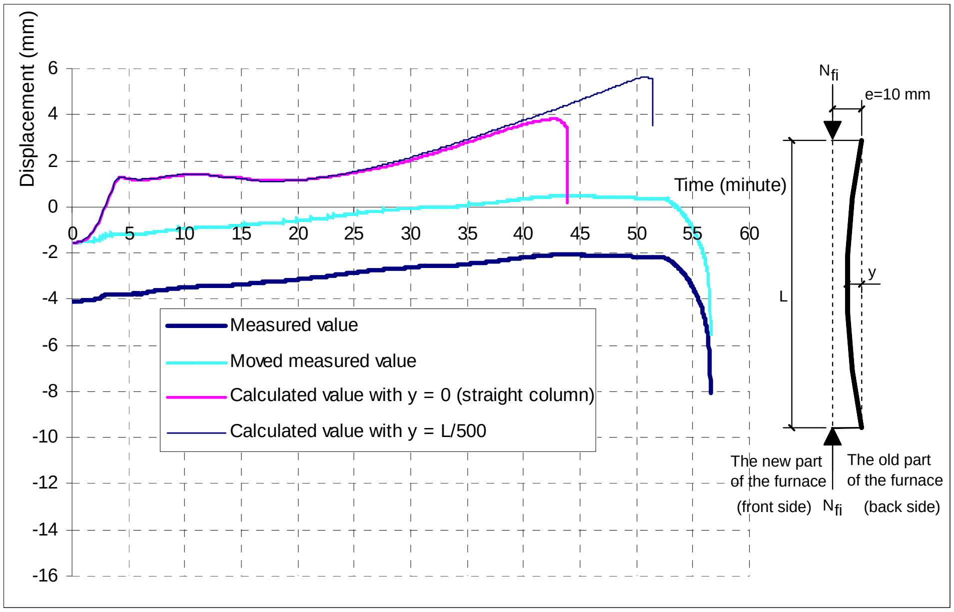


Fig. 12 Vertical displacement of column 3A

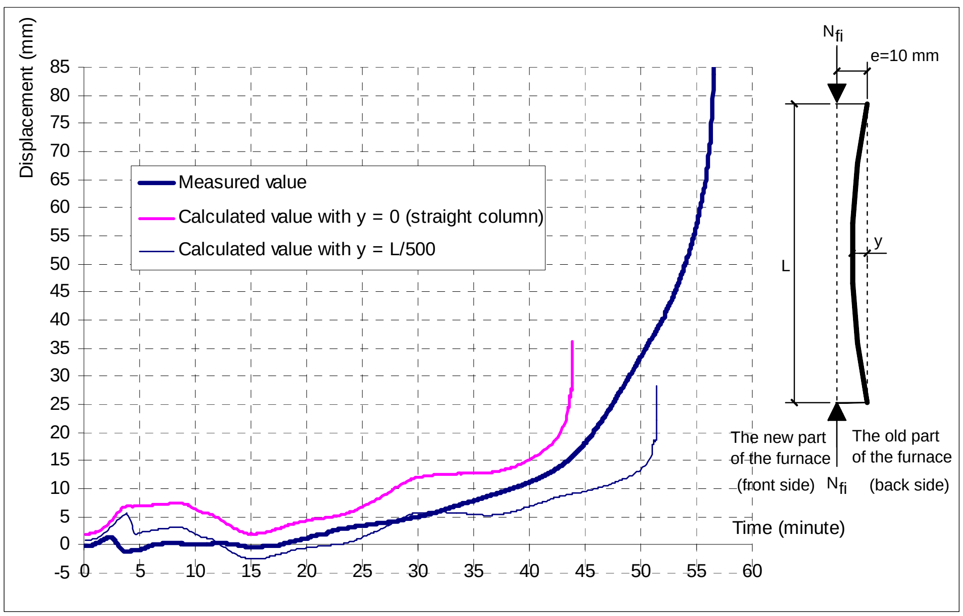


Fig. 13 Transversal displacement of column 3A

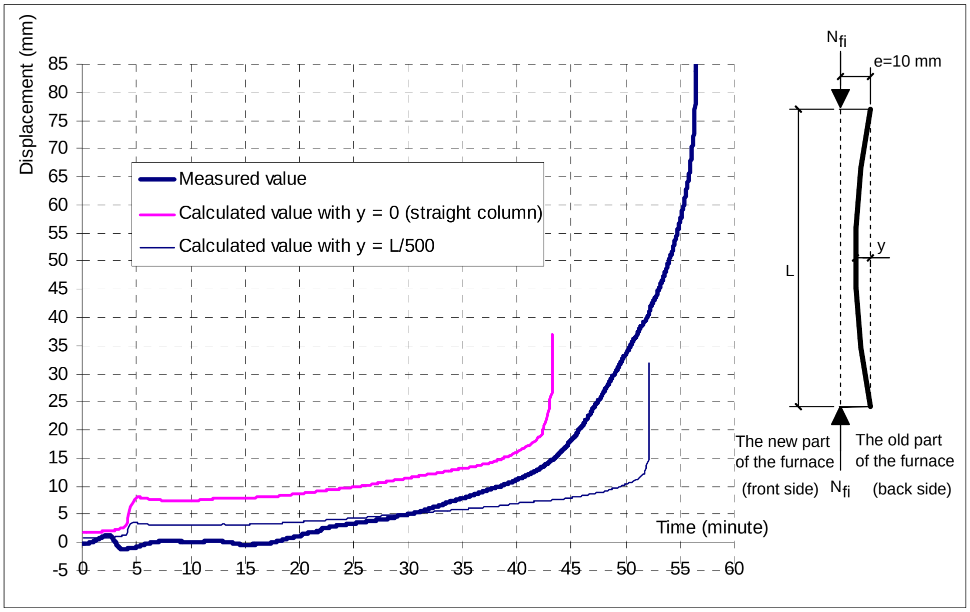


Fig. 14 Transversal displacement of column 3A-
Calculated with uniform temperature in the furnace

## PROCEDURE FOR THE DEVELOPMENT OF A SIMPLIFIED METHOD AND ADDITIONAL SIMULATIONS

Concrete filled hollow steel section (HSS) columns in the fire situation can be designed according to Eurocode 4 – Part 1.2 [6], 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 hollow steel section 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 simplified method for calculating the fire resistance of CFSHS columns has been established in the frame of this research work. Furthermore, the field of applicability has been extended : effective length of column up to 7 m, percentage of reinforcing steel up to 10 % or column embedding another steel profile.

Three steps have been used for the development of the simplified method :

- SAFIR results have been compared with experimental results and some calibrations have been performed;
- A formula for short columns has been established based on SAFIR simulations taking into account the main parameters (quality of materials, dimensions, steel bars, concrete cover);
- The formula has been then extended to slender columns.

More detailed information can be found in reference [13].

In this study, rather small sections have been used with fire resistance Rf 30 or Rf 60. Therefore simulations, not reproduced here, have been performed on larger profiles (dimensions up to 400 mm) in order to reach Rf 90 and Rf 120. Additional data can thus be provided for immediate use by practical engineers.

## CONCLUSIONS

A series of fire tests on steel hollow section columns filled with self-compacting concrete embedding another steel profile has been reported. The following main observations have been made :

- All columns failed by overall buckling;
- Small marks of local buckling of the external steel tube have been observed on some columns;
- The tested columns behaved in a relatively ductile manner;
- The fire resistance is highly dependent on the load ratio.

Numerical simulations using SAFIR computer code have been performed. The following conclusions can be drawn regarding thermal as well as structural analysis :

- The calculated temperatures are in agreement with reality provided a thermal resistance between the steel tube and the concrete core is introduced;
- The value adopted in this study (0.013 m2K/W) has been obtained by numerical experimentation;
- For profiles covered with intumescent paint, a modified thermal conductivity based on a constant thickness of 1 mm has been proposed; its variation during the test has been modelled;
- There is a very good agreement between the tested and calculated values of the fire resistance duration;
- The axial and lateral displacements of the columns are predicted rather well except at the beginning and near failure;
- The ductile behavior near failure could be due to the confinement effect of the internal steel tube not taken into account in the simulations;
- The good predictions of SAFIR regarding fire resistance duration based on the assumption that the properties of SCC are the same as those of normal vibrated concrete constitute an argument justifying this assumption a posteriori.

## ACKNOWLEDGMENTS

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