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STEEL HOLLOW COLUMNS WITH AN INTERNAL PROFILE FILLED WITH SELF-COMPACTING CONCRETE UNDER FIRE CONDITIONS

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Abstract. A detailed experimental and numerical investigation has been performed on the behavior under fire conditions of concrete filled steel hollow section (CFSHS) columns. In this study the internal reinforcement consists of another profile (tube or H section) being embedded with the concrete, and filling is realized by self-compacting concrete (SCC). Ten columns filled with self-compacting concrete embedding another steel profile have been tested in the Fire Testing Laboratory of the University of Liege, Belgium. Numerical simulations on the thermal and structural behavior of these elements have been made using the non linear finite element software SAFIR developed at the University of Liege. There is a rather good agreement between numerical and experimental results, which can be slightly improved by using the ETC (Explicit Transient Creep) model incorporated in SAFIR. This shows that numerical analyses can predict well the behavior of CFSHS columns under fire conditions. The properties at high temperatures of self-compacting concrete are considered the same as those of ordinary concrete.

Keywords: columns, composite steel-concrete construction, fire resistance, hollow steel sections, self-compacting concrete.

1. INTRODUCTION

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 element are now included in codes and standards, like for example in European [5, 6, 7, 8] and American [9, 10] codes.

The classical procedure for realizing this type of profile is to use ordinary concrete to fill the hollow section and to reinforce the columns by classical rebars, mainly to improve the behavior under fire conditions. When the section size is small with respect to the load that the column has to support, because of architectural reasons or in order to maximize the utilization of space in the building, the reinforced concrete filled hollow section may not be sufficient. One possible solution is to include a steel section inside the hollow section. Because of space constraint between the internal section and the hollow section, it may then be necessary to use self-compacting concrete.

In order to analyze the behavior of such columns, it is necessary to investigate the properties of this material at ordinary and elevated temperatures. Many properties related to SCC at ordinary temperatures have been studied (see references in Persson [11]). Another article due to Persson [12] was the first focused on the fire resistance of SCC and is still an excellent reference. But other valuable contributions can nowadays be found in the literature [13, 14, 15, 16].

Very recently an extensive study on the properties of SCC at high temperature by Bamonte and Gambarova [17] has been published. This study confirms that the thermal and mechanical behavior of SCC at high temperature is very similar to that of traditional vibrated concrete.

Ten columns with five different cross-sections have been tested in the Fire Testing Laboratory of the University of Liege.

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.

2. EXPERIMENTAL INVESTIGATION

2.1. Characteristic of the tested columns

A total of ten specimens including eight SCC filled steel hollow section columns without fire protection and two columns protected by intumescent paint have been tested. The test parameters were the cross section type and the load ratio (ratio of the load applied in fire condition to the ultimate load of the member at room temperature).

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.

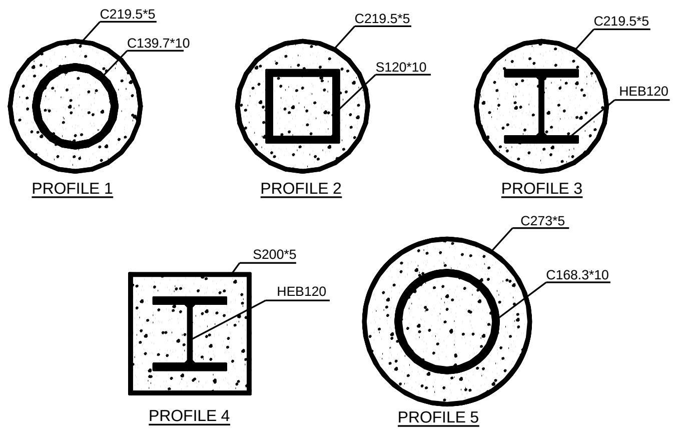

Fig. 1 – Cross-sections of the columns.

Columns 1A and 1B, 2A and 2B, and 5A and 5B are identical but the values of the compressive load are different. The cross-sections 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 than that of column 3B.

The value adopted for the simulation of the compressive strength of self-compacting concrete is 35 MPa. The steel grade of the hollow section tubes and of the HEB profiles is S355.

Table 1

Properties of the tested columns

Test number	External steel profile		Internal steel profile		Ultimate load at normal temp*		Test load in fire		Load ratio
	Dimensions (mm)	Yield strength (MPa)	Dimensions (mm)	Yield strengtl (MPa)	Load Nu (KN)	Eccentricity e (mm)	Load Nfi (KN)	Eccentricity 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 based on the measured mechanical properties at room temperature and assuming that the initial deformation shape of the columns is a semi-sine curve with maximum deflection of L/500.

2.2. Test conditions

The fire tests have been performed in the Fire Testing Laboratory of the University of Liege, Belgium. This facility is accredited according to ISO 17025 [18]. The tests have been carried out in accordance with EN 1365-4 [19].

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. 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.

In all tests, the end conditions were hinged with the displacements in one direction and fixed with respect to the perpendicular direction.

All columns have been subjected to a load situated between 25% and 50% of the ultimate load capacity at room temperature. The values of the load and its eccentricity are detailed in Table 1.

2.3. Main observations during tests

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. 2. Figure 3 shows local buckling that appeared on the steel tube during the test 4B.

It has been observed that all tested specimens behaved in a relatively ductile manner.

Fig. 2 – Column 1A after the test.

Fig. 3 – Local buckling during test.

3. NUMERICAL SIMULATIONS AND ANALYSIS OF THE STRUCTURAL BEHAVIOR

3.1. SAFIR Finite Element Program

SAFIR is a non-linear finite element code developed at the University of Liege [20]. The main purpose of this program is the analysis of structures in the fire condition; ambient temperature conditions can also be considered as a special case. The program, which is based on the Finite Element Method (FEM), can be used to study the behavior of two and three-dimensional structures. SAFIR accommodates various elements for different idealizations, several calculation procedures and various materials. The elements include 2-D and 3-D conductive elements for the thermal analyses, and 3-D BEAM, SHELL and TRUSS elements for the mechanical analyses. The stress-strain material laws are generally linear-elliptic for steel and non-linear for concrete.

Using the program, the analysis of a structure exposed to fire consists of two steps. The first step, referred to as "thermal analysis", involves calculating the temperature distribution inside the structural members. The second step, named "structural analysis", is carried out in order to determine the mechanical response of the structure due to thermal effects, since the external loads are usually assumed to remain constant during the fire or during the fire test.

The transient temperature field within a given network is established by a finite element procedure.

The discretization for plane sections of different shapes is possible by using triangular and/or quadrilateral elements. For each element the material can be defined separately. Any material can be analyzed provided its physical properties at elevated temperatures are known. The variations of material properties with temperature are considered.

3.2. Thermal analysis

All the thermal properties used in this analysis follow the recommendations of the Eurocodes [6, 8].

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 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. 4. The differences are less important than expected (about 30°C to 50°C) but they have been introduced in the simulations.

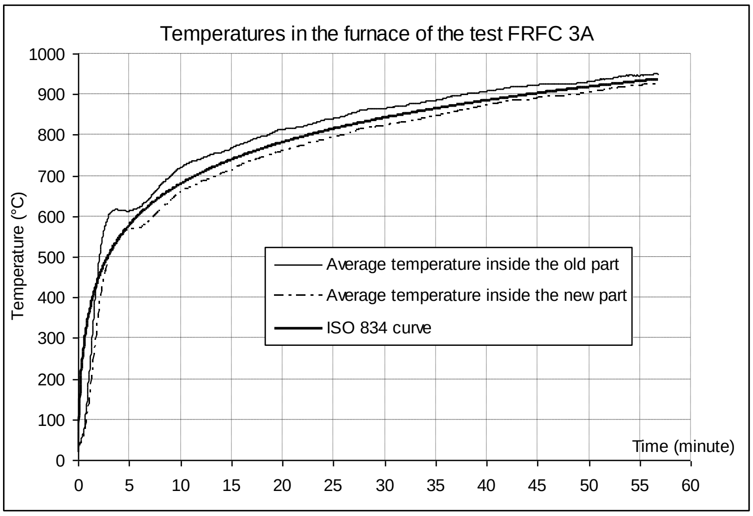

Fig. 4 – Temperatures in the furnace of the test 3A.

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 1 mm in calculations), while the value of the equivalent 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. The models adopted for the modified thermal conductivity of the painted layer of column 3B and 4B can be found in [21].

A comparison between calculated and measured temperatures is shown in Fig. 5 for some particular nodes of column 3A. It can first be seen that one of the thermocouples failed during the test. The calculated temperatures on the external hollow section are in good agreement with the measured temperatures (Fig. 5).

These differences can be explained by the migration of vapor in concrete (not taken into account in the model). 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 temperature from 100 °C to 150 °C.

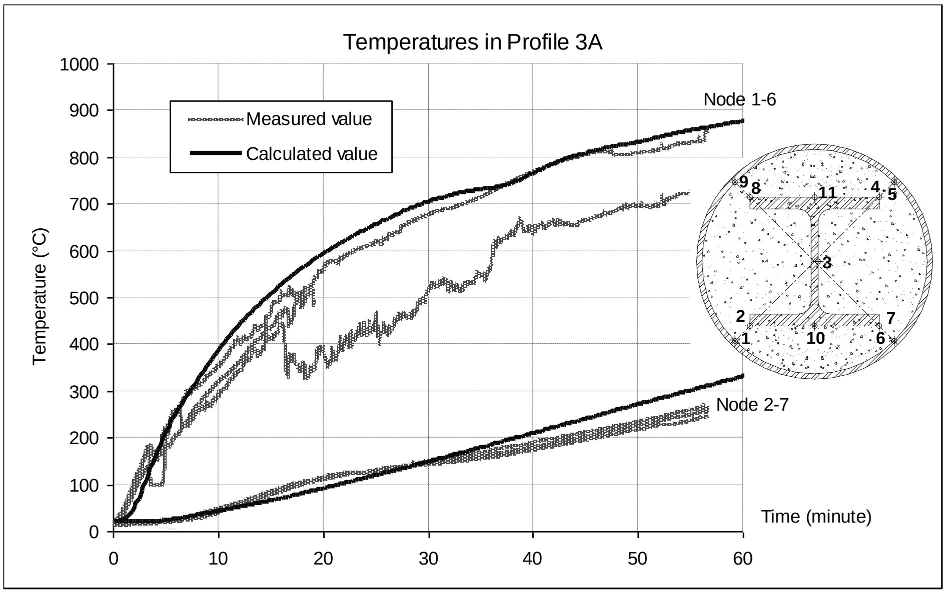

Fig. 5 – Temperatures in the section of column 3A.

3.3. Structural analysis using Eurocode models [6, 8] for the material properties

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

Table 2

Comparison between calculated and measured fire resistance of tested columns

Test number	Test load in fire		Load ratio	Tested	Calculated	
	Load	Eccentricity		fire resistance	fire resistance	R_{t_cal}/R_{t_test}
	Nfi (KN)	e (mm)	Nfi/Nu	$R_{t\text{-}test}$ (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

The calculated lateral displacements are compared to the measured ones for all tested columns. The result for column 3A is shown in Fig. 6.

As can be seen the lateral displacement of the columns is sensitive to the assumed initial deformation. The simulation is better when assuming $y = L/500$ than when assuming $y = 0$ (no initial deformation).

In most tests, after 2 to 4 minutes of fire, the columns deform toward the old part of the furnace and after some time toward the new part (Fig. 6). This phenomenon does not appear in simulations if a uniform temperature around the column is adopted. If the thermal gradient measured in the furnace is introduced, the calculated lateral displacement curve has the same form as the recorded one (Fig. 6). Therefore, the differences in the lateral 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 is assumed in the simulations to predict the fire resistance only.

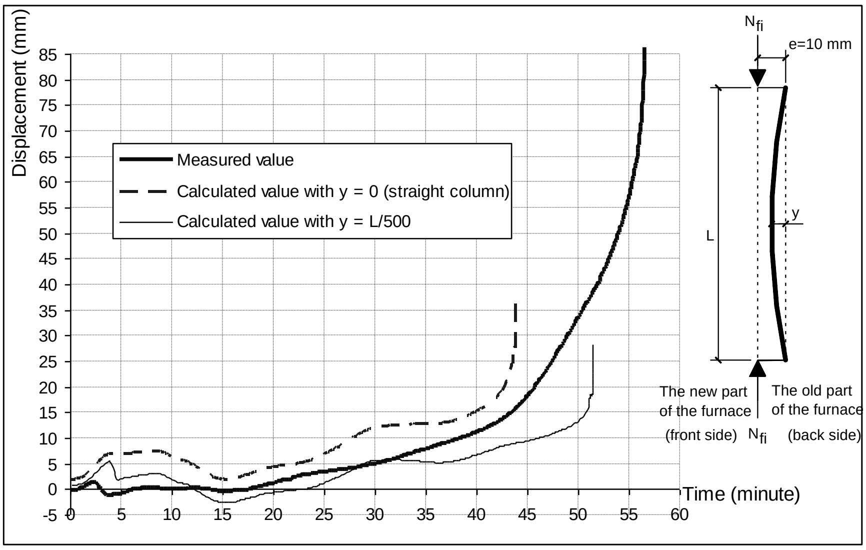

Fig. 6 – Lateral displacement of column 3A – calculated with EC2 model and a thermal gradient in the furnace.

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. This ductile behavior is probably due to the transversal effects between the concrete and the steel profiles (internal and external). This analysis is beyond the scope of the present research. The study of this parameter in CFSHS columns like those examined here could be a perspective for additional research works.

3.4. Structural analysis using the ETC (Explicit Transient Creep) model

The numerical simulation of column 3A has been run again using the Explicit Transient Creep (ETC) Eurocode model, developed at University of Liege [22] for modeling the thermo-mechanical behavior of concrete. This ETC model includes an explicit term for transient creep strain in the strain decomposition, whereas in the current Eurocode model, the effects of transient creep are incorporated implicitly in the mechanical strain term.

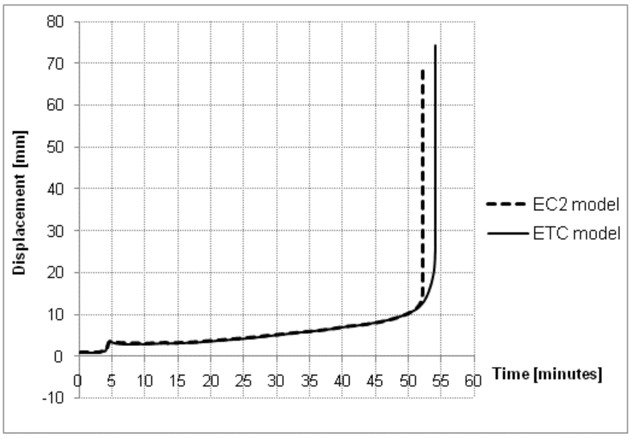

Fig. 7 – Column 3A – calculated with a uniform temperature in the furnace. Effect of the concrete material model used in the numerical simulation on the computed results

In concrete, transient creep strain develops irrecoverably during first-time heating of concrete under load, compared to concrete loaded at elevated temperature [23, 24]. This strain has to be considered in any fire analysis involving concrete in compression, as any stress analysis of heated concrete which ignores transient creep will provide erroneous results [25].

The main benefit of the explicit approach adopted in the ETC model over the implicit approach of Eurocode is the ability to take into account the influence of the stress-temperature path on the strain response [26]. Fig. 7 shows the effect of the concrete material model used on the computed results.

4. CONCLUSIONS

A series of ten 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;

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 analyses:

For the eight tests without thermal protection, the calculated temperatures are in reasonable agreement with the measured temperatures provided that a thermal resistance of 0.013 m^2K/W is introduced between the steel tube and the concrete core; for the two tests with thermal insulation, the thermal conductivity of the painted layer has been transformed into an equivalent conductivity based on a constant thickness of 1 mm to lead also to good agreement.

There is a very good agreement between the measured and calculated values of the fire resistance duration;

The lateral displacements of the columns are predicted rather well except at the beginning of the test and near time of failure provided that an initial deformation of L/500 is adopted;

The simulations performed with the new ETC (Explicit Transient Creep) model developed at the University of Liege lead to values closer to experimental results regarding the fire resistance duration as well as the lateral displacements of the column near failure. Nevertheless this cannot explain completely the amount of ductility shown in the test results. In order to capture this latter phenomenon, new simulations should be performed taking into account the triaxial effects occurring inside the column. To this aim the ETC model has to be completed, as has been shown in the associated research study [27].

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, which is in agreement with other research works.

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