# Numerical simulation of steel and composite steel and concrete columns in fire

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ABSTRACT: The design of structures in fire is a subject of great importance in the project and construction of buildings, not only because it guarantees the maintenance of a scenery of structural stability of the building for the people's evacuation and fire combat by the firemen, but also because it can in the limit preserve the structural integrity of the building after fire. The majority of the structural codes for fire design (e.g., Eurocodes) they indicate relatively simple methods for fire design of single elements, that don't consider all phenomena involved in fire situation, however they open to the designer the possibility of using advanced calculation methods where the structure may be calculated as a whole. The present paper presents the comparison of results of numerical simulations carried out with an advanced calculation method and fire resistance tests performed in the University of Coimbra on steel and composite steel and concrete columns with elastically restrained thermal elongation. The advanced calculation method is the finite elements program SAFIR.

## 1 INTRODUCTION

The fire resistance of steel columns has been studied for years. A lot of numerical and experimental tests have been performed on single steel columns. However, the number of tests on composite steel and concrete columns is still very a few.

In the last three years, in the Laboratory of Testing Materials and Structures of the Department of Civil Engineering of the Faculty of Sciences and Technology of the University of Coimbra, a large experimental program on the fire resistance of steel and composite steel and concrete columns with restrained thermal elongation was carried out. The results of the experimental tests were simulated using the finite element program SAFIR, actually in development in the University of Liége, in Belgium, by Jean Marc-Franssen (2005).

The results are analysed and several conclusions on the behaviour of steel and composite steel and concrete columns in fire are outlined (Carvalho 2008).

## 2 EXPERIMENTAL TESTS

In the experimental tests, a new set-up for fire resistance of building columns with restrained thermal elongation was developed (Figure 1). This system allowed studying the influence of several parameters that have influence in the behaviour of building columns in fire, as for example: the stiffness of the surrounding structure, the slenderness of the column, the load level and the type of cross-section.

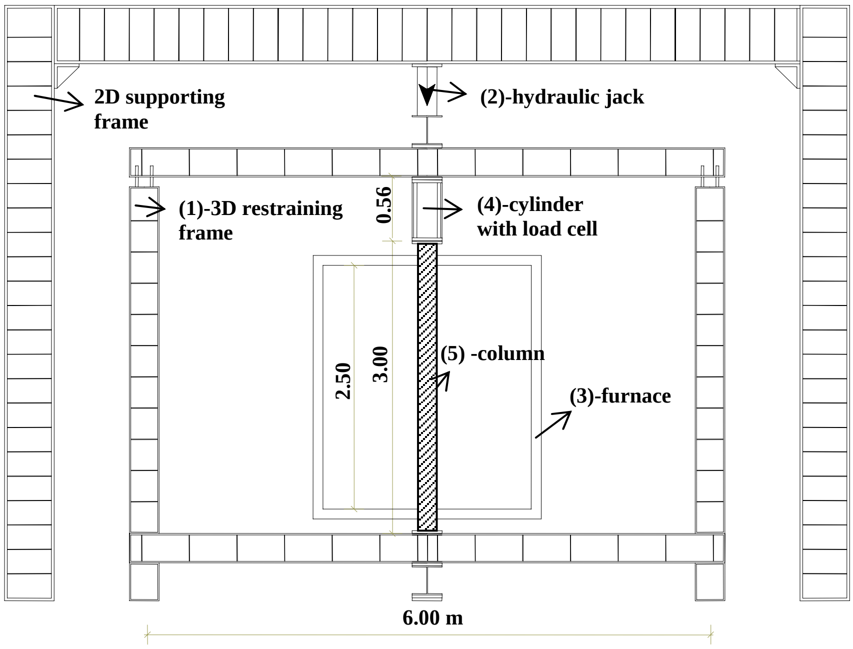


Figure 1. Scheme of the test set-up for fire resistance tests on columns

The experimental system comprised a 3D restraining frame (1) of variable stiffness with the function of simulating the stiffness of the surrounding structure to the column in fire.

The columns were subjected to a constant compressive load of 70% of the design value of buckling resistance of the column at room temperature (Eurocode 3 – part 1.1 2005, Eurocode 4 – part 1.1 2004). This load was controlled by a load cell of 1MN located on the head of the hydraulic jack (2). This load simulated the serviceability load of the column when part of a real structure. The load was applied by the hydraulic jack (2) controlled by a servo hydraulic system.

The thermal action was applied by a modular electric furnace (3) that could closely follow the ISO 834 fire curve (Eurocode 1 – part 1.2 2002).

The restraining forces generated in the column due to heating were measured by a load cell of 3MN located into a void steel cylinder (4). This cylinder was placed between the testing column (5) and the restraining frame (1).

The axial displacements and rotations on the top and base of the column were also measured by displacement transducers orthogonally arranged in three different points, forming a deformation plane.

The temperatures were registered by thermocouples type K in five sections of the steel and composite steel and concrete columns tested (Figure 2). The thermocouples were welded in the steel profile for the case of the steel columns and welded in steel profile and longitudinal reinforcing bars and embedded in the concrete for the case of the composite sections. The thermocouples in the concrete were at different depths allowing like this to know the gradient of temperatures in the cross-section.

The fire resistance tests were carried out on steel and composite steel and concrete columns with cross-sections of HEA160 and HEA200.

In the composite steel and concrete columns the HEA160 and HEA200 sections had longitudinal reinforcement bars of 16mm and 20mm of diameter, respectively.

The testing columns were rigidly connected to the restraining frame by four M24 bolts of class 6.8.

The stiffness of the restraining frame to thermal elongation of the column tested was about 3, 13, 39 and 68 kN/mm. These stiffness ratios were measured in experimental tests at room temperature specially carried out for those propose.

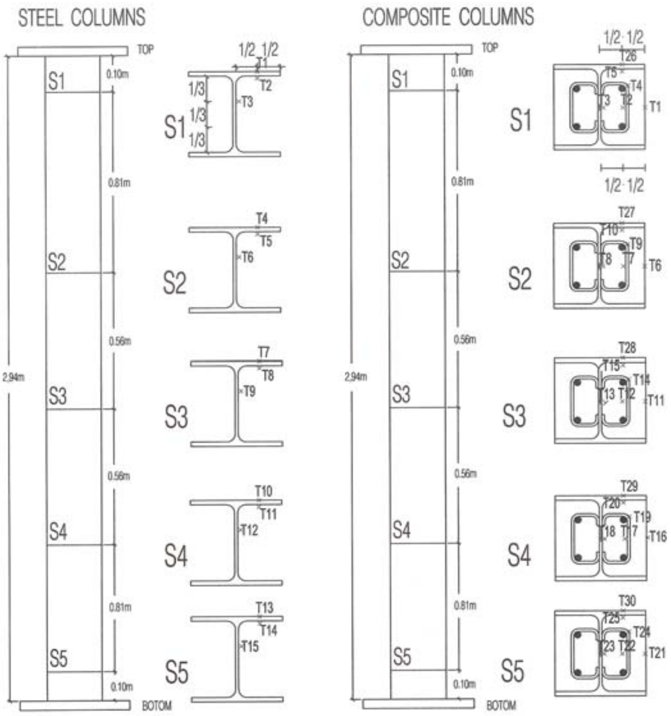


Figure 2. Location of thermocouples in the columns

## 3 NUMERICAL SIMULATIONS

The process of calculation of SAFIR bases on two stages, a thermal analysis followed by a mechanical analysis of the elements. The thermal analysis allows the determination of the temperature distribution while the mechanical analysis allows the determination of the forces, moments, stresses and strains in the cross-section. The program also enters into account with the variation of the mechanical and thermal properties of the materials in function of the temperature (Franssen 2005).

The conditions of the experimental tests were reproduced in SAFIR as most rigorous as possible. The 3m column tested was discretized in 14 beam elements with variable lengths, between 100 mm and 280 mm. In the upper part of the column the cylinder to measure the restraining forces was simulated by a solid cylinder of steel measuring 565 mm in length and 300mm in diameter (Carvalho *et al* 2009).

The beams and columns of the restraining frame HEB300 that were used to simulate the stiffness of the surrounding structure of the column, were discretized in 73 beam elements with lengths that varied between 375 and 935mm.

The temperatures considered in the numeric simulations were the ones registered in the experimental tests. In the steel cross-section it was considered a uniform distribution of temperatures, since the gradient of temperatures for these sections was very small. However, for the composite steel and concrete cross-sections, the variation of the temperatures in depth was considered, being the cross-section divided in small areas of equal temperature.

The steel of the columns tested as well as the steel of the elements of the restraining frame was of class S355 (Eurocode 3 – part 1.1 2005) while the concrete of composite columns was of class

C25/30 (Eurocode 2 – part 1.1 2004) and the steel reinforcement bars A500NR. The bolts of the restraining frame were all of class 8.8.

The mechanical and thermal properties of the materials in function of the temperature used in the numerical simulations are in EC1, EC3 and EC4 parts 1.2 (Eurocode 2 – part 1.2 2004, Eurocode 3 – part 1.2 2005, Eurocode 4 – part 1.2 2005).

The initial axial loads applied to the columns in the numerical simulations are presented in table 1.

Table 1 – Initial loads

| **steel columns** | | **composite columns** | |
|---|---|---|---|
| **HEA 160** | **HEA 200** | **HEA 160M** | **HEA 200M** |
| 640 kN | 1025 kN | 555 kN | 972 kN |

The stiffness of the restraining frame to thermal elongation of the column was 13kN/mm.

### 3.1 *Evolution of temperatures*

Figures 3 and 4 show the evolution of temperatures in the steel cross-section $S_3$, the furnace and ISO 834 target temperatures. In these figures, as expected a uniform temperature distribution in the cross-section is observed. The fire resistance test was very short for that it had necessary to adjust the curve of the furnace, as best as possible, to the ISO 834 fire curve.

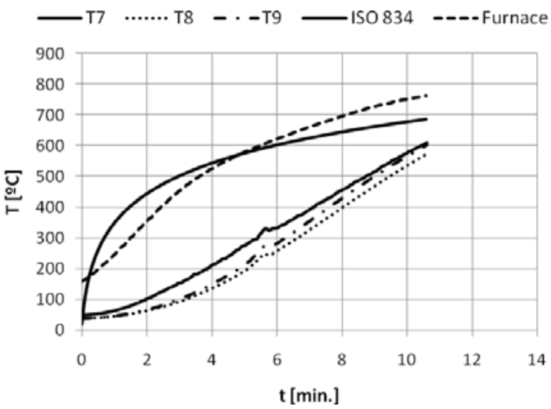


Figure 3. Temperatures in steel cross-section HEA160

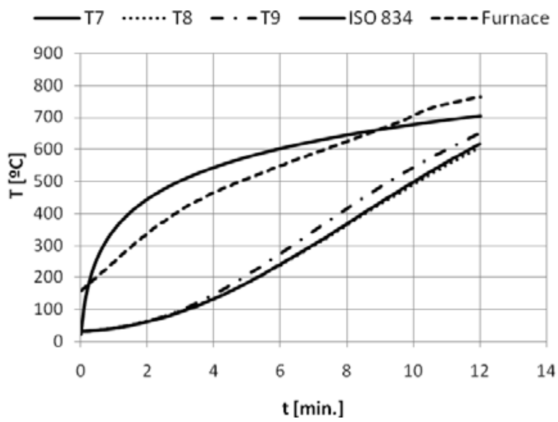


Figure 4. Temperatures in steel cross-section HEA200

Figures 5 and 6 show the evolution of temperatures in the composite steel and concrete cross-section $S_3$, the furnace and ISO 834 target temperatures. In these figures, it is observed that the evolution of temperatures in concrete, although uniform, is much smaller than in the steel.

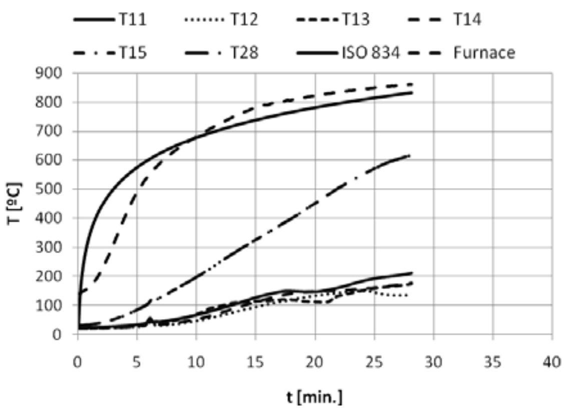


Figure 5. Temperatures in composite cross-section HEA160M

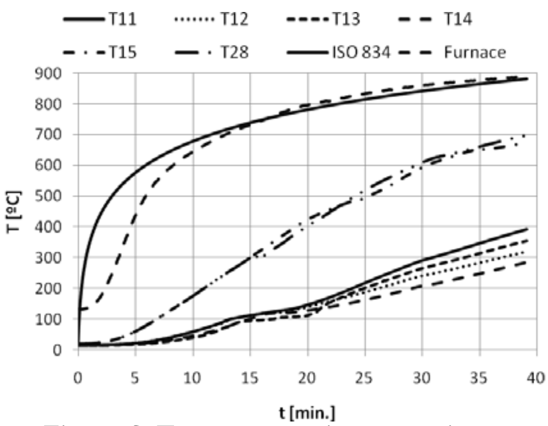


Figure 6. Temperatures in composite cross-section HEA200M

Figure 7 shows the evolution of temperatures in height for the tested HEA160 and HEA200 steel columns, for 3, 5 and 10 minutes of fire duration. It is observed an increasing of the thermal gradient in height in function of the time. For the same instant of time the temperatures in the HEA160 are higher than in the HEA 200 columns.

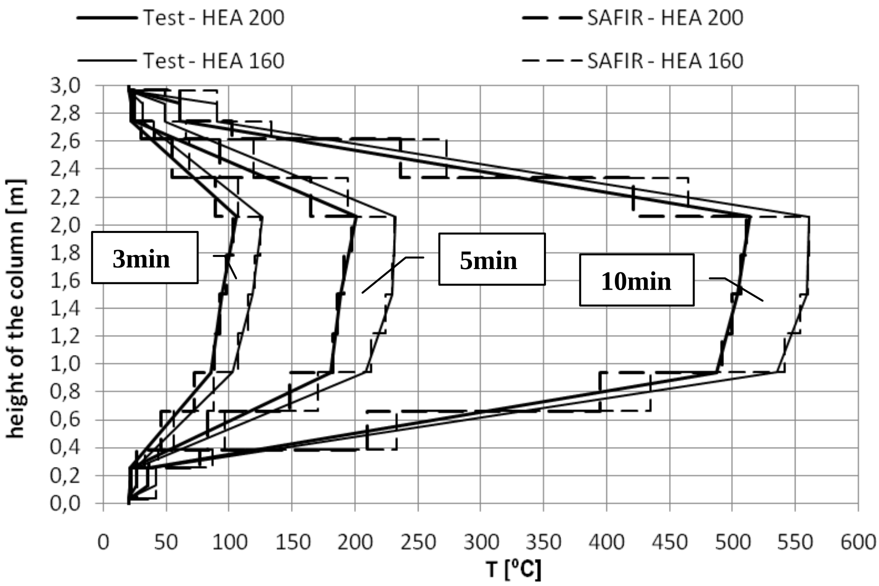


Figure 7. Distribution of temperatures in height for HEA160 and HEA200 steel columns

## 3.2 *Restraining forces*

Figures 8 and 9 present the evolution of restraining forces related to the initial load in function of the steel mean temperature of the column and the time, respectively. In these figures they can be observed that some difference of values exists, between the experimental tests and the numeric simulations, although the global development of the curves is similar.

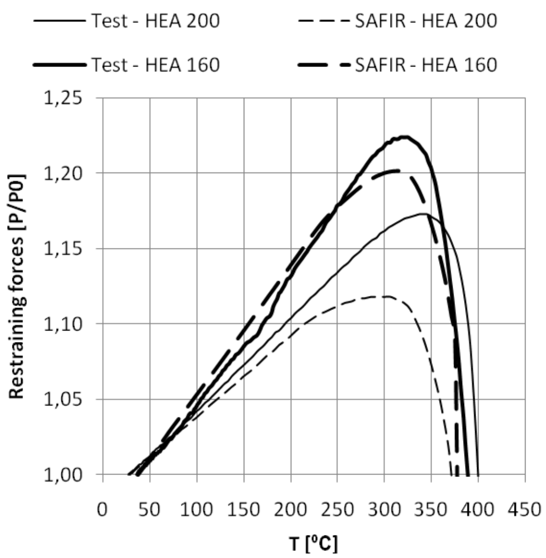


Figure 8. Restraining forces *vs* steel mean temperature for steel columns

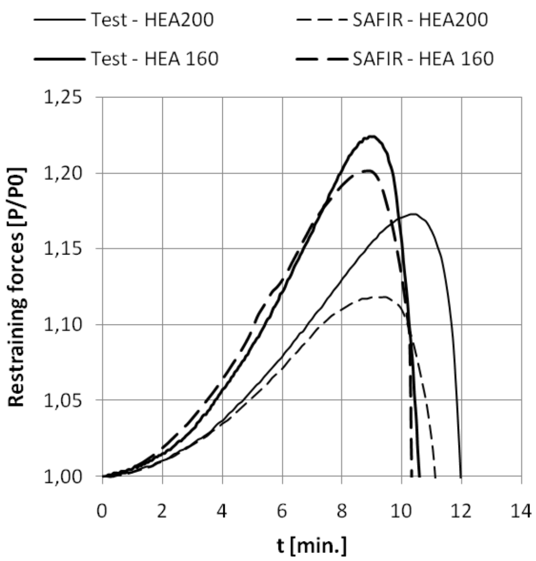


Figure 9. Restraining forces *vs* time for steel columns

The critical temperature in both sections is nearly the same for the tests and numerical simulations, around 400ºC that corresponds to 11min. (Figure 9).

Comparing figures 9 and 10, the maximum restraining forces in percentage registered in the composite columns is smaller than in the steel columns for the same type of steel profile.
The instant of time correspondent to the critical temperature is similar for both steel columns the same is not observed for the composite columns, where the HEA200 section presents higher fire resistance.

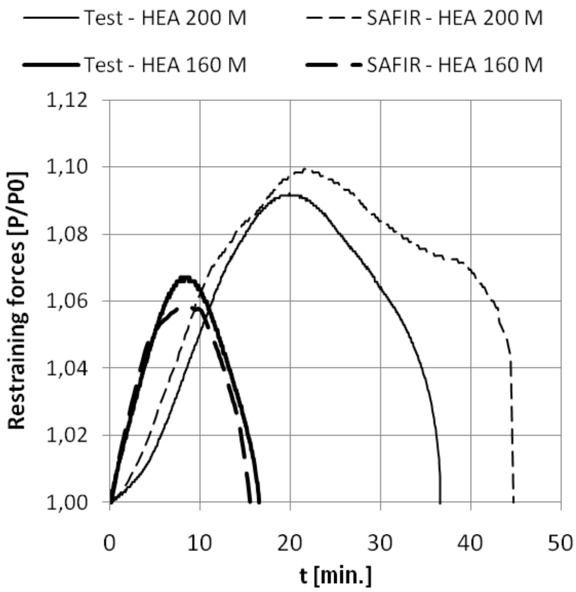

Figure 10. Restraining forces *vs* time for composite columns

## 3.3 *Axial displacements and rotations*

In figures 11 and 12 some discrepancy can be observed between the results of the numerical simulations and the experimental tests, although the results of the numerical simulations are very close to each other as well as the results of the experimental tests.

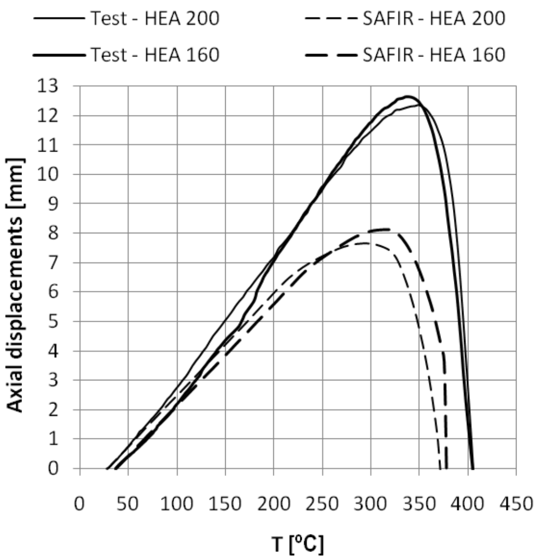

Figure 11. Axial displacements *vs* steel mean temperature for steel columns

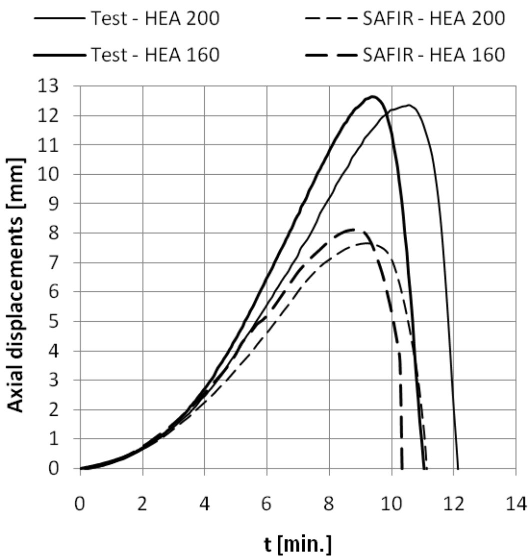

Figure 12. Axial displacements *vs* time for steel columns

For the case of the HEA160 section, the maximum axial displacements were about 12.6 mm in the tests and 8.1 mm in the numerical simulations. For the case of the HEA200 section, the maximum axial displacements were about 12.2 mm for the test and 7.7 mm for the numerical simulation. A difference in average of 4.5 mm between the numerical simulations and the experimental tests is observed. This difference is maybe explained by some plays in the experimental system or parameters not well modelled in the numerical simulations.

In figures 13 and 14 the development of the rotations is presented in function of the steel mean temperature and the time, respectively. In the figures they can be verified that the rotations suffered similar developments in the tests and in the numerical simulations, with values of order of greatness slightly different, mainly in later phases of the tests. The rotations in the numerical simulations are much smaller than in the experimental tests.

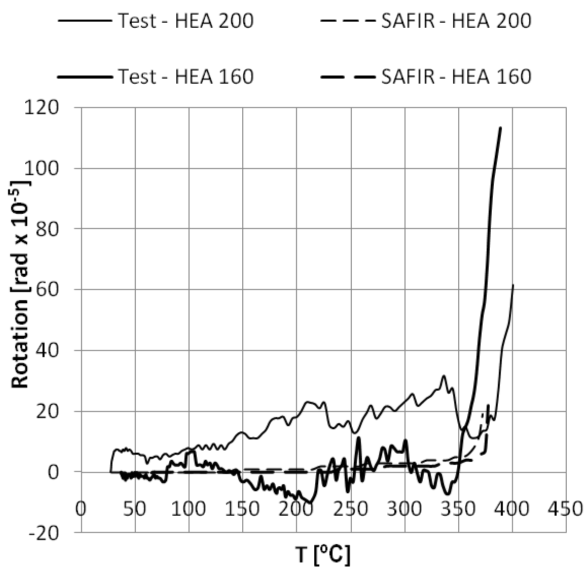


Figure 13. Rotations in top of the steel columns *vs* mean steel temperature

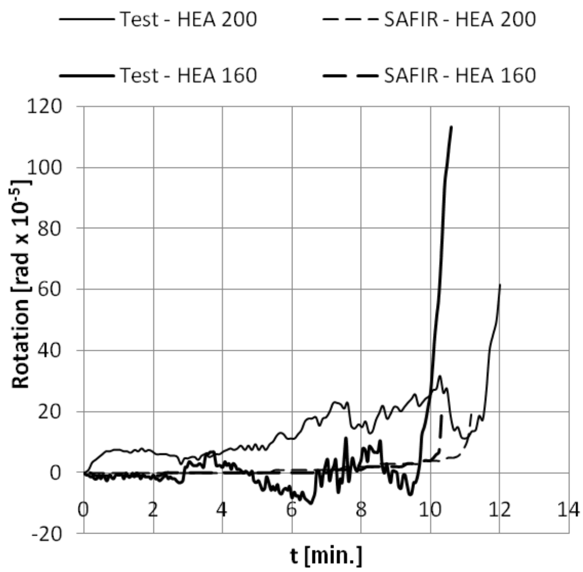


Figure 14. Rotations in top of the steel columns *vs* time

In Figure 15 the axial displacements for the composite columns in function of time are plotted. In this case the displacements between the numerical simulations and the experimental tests are closer to each other than in the steel columns.

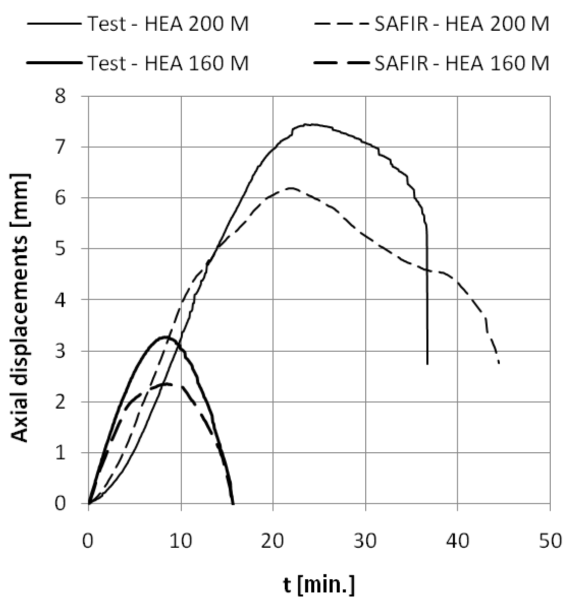


Figure 15. Axial displacements *vs* time for composite columns

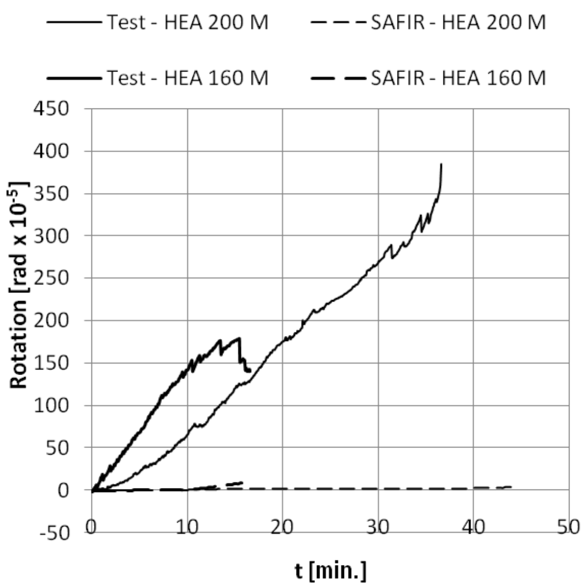


Figure 16. Rotations in top of the composite columns *vs* time

For composite steel and concrete columns big differences in the rotations between the numerical and the experimental tests are observed (Figure 16). These differences may arise due to the fact that the rotational stiffness of the restraining frame to the column in the experimental tests was maybe not well modelled in the numerical simulations.

## 4 CONCLUSIONS

The numerical simulations using the program SAFIR described the experimental tests quite well. The differences observed in the restraining forces and in the axial displacements possibly results from the model used and the assumptions made in the numerical simulations for the real test conditions. Some of the differences could be due to the fact that the model cannot correctly foresee the axial and rotational stiffness of the ends of the column.

It should be also highlighted that in these experiments, there were a large number of factors that could disturb the results. Factors such as the deviations of straightness of the elements and measurement errors of the sensors are not possible to simulate numerically.

An interesting conclusion of this study is that the composite sections present a higher difference in the fire resistance when changing the dimensions of the steel profile than in the case of the steel sections. The concrete has a great influence in the fire protection of the steel profile.

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