Steel and Composite Structures, *Vol. 17, No. 6 (2014) 835-850*
DOI: http://dx.doi.org/10.12989/scs.2014.17.6.835

Fire design of concrete encased columns: Validation of an advanced calculation model

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(*Received December 15, 2012, Revised December 03, 2013, Accepted April 29, 2014*)

Abstract. The fire resistance of composite steel and concrete structures may be determined by using the simplified methods provided in EN 1994-1-2. For the particular situations not covered by the standard, an advanced calculation model might be applied, using special purpose programs for the analysis of structures in fire. The validation of these programs has always been an important issue for software developers, but also for designers and authorities. Clause 4.4.4 from EN 1994-1-2 refers to the validation of the advanced calculation models and states that these models must be validated through relevant test results. The paper presents the calculation of fire resistance of the composite columns in a high-rise building built in Romania, and focusses on the validation of the calculation model (computer program SAFIR), for this particular case. This validation, asked by the Romanian authorities, considers the available experimental results of a fire test, performed on a similar composite steel-concrete column.

Keywords: composite columns; fire design; advanced calculation model; validation; fire test

1. Introduction

One challenging aspect in fire design is to determine the temperature distribution inside the cross-section of the structural elements. Once the temperature field is determined, the resistance of the member may be calculated using a classical integration on the depth of the cross-section, considering the reduced material characteristics, function of the temperature.

For steel elements, EN 1993-1-2 (2005) offers a simple method based on the section factor, to determine a unique temperature within the cross-section of the profiles. For composite steel-concrete elements, due to the massivity of the cross-sections, this approach is not possible. Considering also the limitations of the simple calculation models provided in EN 1994-1-2 (2005), it is often necessary to use advanced calculation models in order to evaluate, at least, the temperature distribution inside the cross-section. Even EN 1994-1-2 (2005) states, in Clause 4.4.1, that "compared to tabulated data and simple calculation models, advanced calculation models give an improved approximation of the actual structural behaviour under fire calculations".

According to EN1994-1-2 (2005), the advanced calculation methods may include separate calculation models for the determination of the development, the distribution of the temperature within the structural members (thermal response model) and for the mechanical behaviour of the

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http://www.techno-press.org/?journal=scs&subpage=8 ISSN: 1229-9367 (Print), 1598-6233 (Online)

structure, or of any part of it (mechanical response model). It is interesting that EN 1994-1-2 uses, in the above sentence, "may include", while EN 1992-1-2 (2005) and EN 1993-1-2 (2005) use "should include". In fact, there is no reason why a model that would perform a fully coupled thermo-mechanical analysis could not deserve the title of advanced model (Franssen *et al*. 2009).

Advanced calculation models may be associated with any heating curve if the material properties are known for the relevant temperature range, and may be applied for any type of cross-section. Advanced calculation models for mechanical response shall be based on the acknowledged principles and assumptions of the theory of structural mechanics, taking into account the changes of mechanical properties with temperature. If the stress-strain relationships given in EN 1994-1-2 (2005) for the materials are used, the effects of high temperature creep need not to be considered. This creep is supposed to be incorporated implicitly in these relationships.

EN 1993-1-2 (2005) states that for the analysis of isolated vertical members, a sinusoidal initial imperfection should be used, with a maximum value of 1/1000 of the member length at mid-height, if not specified otherwise by relevant product standards. There is no information about an initial imperfection, when analysing isolated concrete or composite members in EN 1992-1-2 (2005) and EN 1994-1-2 (2005).

Clause 4.4.4 of EN1994-1-2 (2005) refers to the validation of the advanced calculation models, implemented in computer programs for the structural analysis under elevated temperatures.

The verification and the validation of the numerical models used in structural engineering represent a very important aspect, in order to assess their capability in accurate simulations of the real phenomena. The importance of well-documented benchmarks for verification and validation of the computer software was emphasised by different authors, as Oberkampf and Trucano (2008). The verification of a numerical model means the comparison with analytical solutions or with other computer codes. In principle, the validation is based on the comparison with experimental results.

The soundness of an experiment as a source of data for validation depends on the relationship between the application and the validation domains (Oberkampf *et al*. 2004), the ideal situation being when the two domains overlap. The application domain rests in the boundaries of the predictive capabilities of the numerical model, while the validation domain characterizes the representation capabilities of the experiment. In the analysis of complex systems, it is sometimes infeasible or even impossible to conduct all necessary experiments to verify all features of the computational model (Kwasniewski 2009). On the other hand, not only the number of available experiments is limited, but also the usable results of the tests that may be collected in the literature, due to the incomplete information about the input data, support conditions or of the results. It may also happen that the numerical simulation reveals significant disagreements in comparison with the experiment; in such situations, the reasons for this should be sought in both sides (Kwasniewski 2009).

Finite element codes, suitable for structural-fire analyses, can be broadly divided into two categories (Gillie *et al*. 2008): commercial general-purpose codes, such as ABAQUS, ANSYS, etc. and research-based codes such as VULCAN or SAFIR, to cite just two of the most known ones. In literature, there are many references regarding the comparisons of the numerical analyses, which use these programs against fire tests on steel-concrete members or even structures.

The numerical investigation of the structural results from a compartment fire test, conducted in 2003 on the full-scale multi-storey composite building constructed at Cardington, U.K. (STC, 1999), was performed by Foster *et al*. (2007), using the computer program VULCAN, developed at the University of Sheffield, which offered good results in comparison with the experimental ones. More recently, a three-dimensional eight-node brick element, capable of representing the performance of composite structures subjected to 3D stress conditions at ambient and high temperatures, has been incorporated in the same research based code VULCAN. The model was validated against the results from a number of tests on composite structures subjected to 3D stress conditions, at both ambient and elevated temperatures (Yu *et al*. 2010).

Fire design using advanced calculation models may consider the fire action applied to the structural members in a separate thermal analysis, following a nominal or a natural fire model, as given in EN 1991-1-2 (2005). Another modern approach is to use an integrated fire dynamics and thermo-mechanical modelling framework, as done by Choi *et al*. (2010), who validated their numerical simulations using ABAQUS, through one of the six Cardington compartment tests on the full-scale steel-concrete composite structure, above mentioned.

In order to assess the behaviour of steel or composite connections in fire, parametric studies are necessary. A parametrical experimental study would be an expensive solution, and, therefore, the typical approach is to perform a parametric numerical analysis, after an appropriate calibration and validation of the numerical model against available experimental results. Jones and Wang (2008) made such an approach, by validating a numerical model in ABAQUS through an experimental program for welded fin-plate connections of hollow and concrete filled tubular columns, at ambient and elevated temperatures.

The computer program SAFIR (Franssen 2005) is widely used, and is internationally recognised as a special purpose computer program for the structural analysis in fire conditions, for both research and design purposes. SAFIR satisfies the conditions of the fire parts of Eurocodes for an advanced calculation model, and implements the thermal and mechanical characteristics of the materials provided in these norms. In the last two decades, SAFIR proved its capability to reproduce the fire behaviour of steel, concrete and steel-concrete composite structures against experimental results. Just a few references may be mentioned here: a 2D analysis to model an early Cardington fire test performed in 1987 on a full-scale steel frame (Franssen *et al*. 1995), and some more recent 3D analyses, to model the behaviour of cellular composite beams tested as isolated elements (Nadjai *et al*. 2007) or the membrane effect of slabs in fire, using a full scale test on a composite steel-concrete slab under natural fire (Vassart *et al*. 2011).

Clause 4.4.4 of EN 1994-1-2 (2005) states that the validity of any computer program that uses an advanced calculation model for the analysis of structures under elevated temperatures, shall be verified on basis of *relevant* test results. The keyword, in this clause, is "relevant". When using a computer program for calculating the fire resistance of a structural member or of a structure, the designer should be aware that the program is able to reproduce the behaviour of a specific category of applications (for instance composite steel-concrete structures). Once the computer program was validated for a specific problem it is obvious that it is not necessary to perform a fire test for the same type of structural element, in a given design situation. The validation is made for a calculation model (computer program), so that it can be applied for a range of applications. In fact, this is the reason to use a computer program for fire design; otherwise, a fire test for each member of the analysed structure would be made, with the corresponding length, cross-section dimensions and loading. This would be not only uneconomical, but also practically impossible.

Clause 4.4.4 of EN 1994-1-2 (2005) also mentions to perform a sensitivity analysis, in order to verify that the calculation model complies with the sound engineering principles. There are some examples for the critical parameters of this sensitivity analysis, such as buckling length, size of elements and load level.

Even if a special purpose computer program that implements an advanced calculation model for fire structural analysis has already been validated for a category of applications, it is, of course, possible to perform a supplementary validation of the program and a corresponding sensitivity analysis, if experimental tests on elements that are similar to a particular design situation exist.

The Romanian authorities (General Inspectorate for Emergency Situations) requested such a particular validation, in order to demonstrate the ability of the advanced calculation model SAFIR to reproduce the fire behaviour of partially concrete encased columns with crossed I-sections. The authors used SAFIR to determine the fire resistance of the columns of this particular type of columns of “Bucharest Tower Center” building situated in Bucharest, Romania. Because the validation of advanced numerical models used for fire safety engineering has always been a concern of the authorities and because a fire test of a column with cross-section similar to the columns of the “Bucharest Tower Center” was available, the verification of the results given by SAFIR against this test was required.

The paper presents the results of the numerical fire resistance assessment made by the authors for the composite steel-concrete columns of “Bucharest Tower Center” building and the validation of the advanced calculation model SAFIR for this particular design situation, according to the clauses of EN 1994-1-2 (2005).

2. Structural and fire safety aspects of the building

The architect of the „Bucharest Tower Center” office building was “Westfourth Architecture PC/SA”, while the structure was designed by “Britt Ltd.” and “Popp şi Asociaţii Ltd.”. The calculation of the fire resistance of the composite steel - concrete columns was performed at the Politehnica University of Timisoara, Romania.

The building is located in Bucharest, on the Boulevard Ion Mihalache, one of the convergent streets in Victoria Square, in which the monumental buildings of the Natural Science Museum and the Geological Museum, built in the 19th century, and the Palace of the Government, contrast with the new buildings.

The building has 4 underground levels, ground level, 22 stories and 3 technical levels. The ground floor built surface is of 1434 m^2, the first to the third upper floors are of 1859 m^2 and the floors 4-22 have a surface of 800 m^2 (with a small enlargement at floors 7-10 for aesthetic effect).

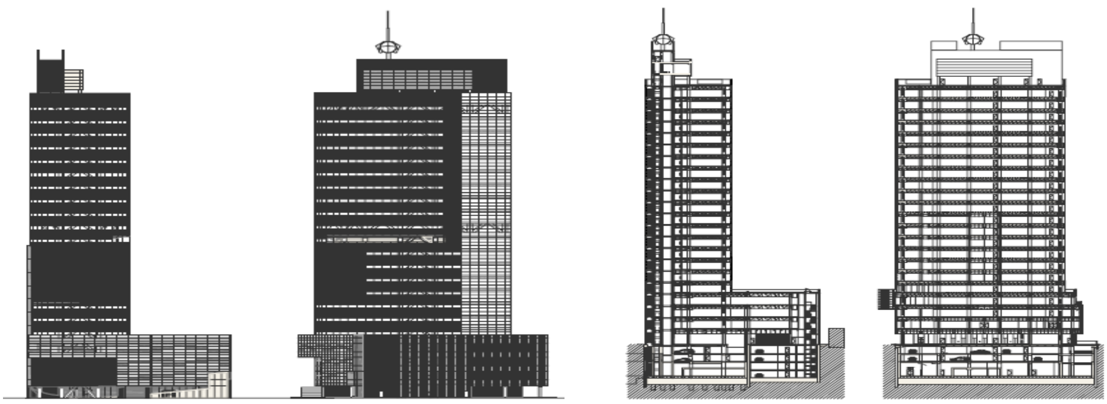


Fig. 1 Facades and sections of “Bucharest Tower Center” office building

Fig. 2 The building during construction

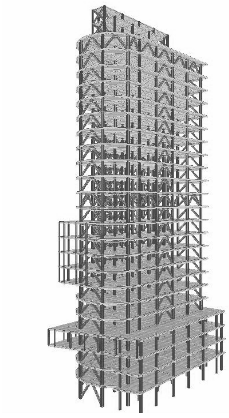
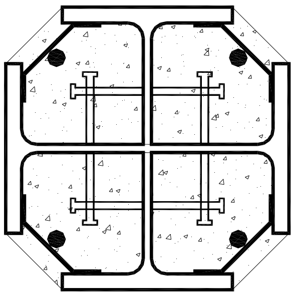
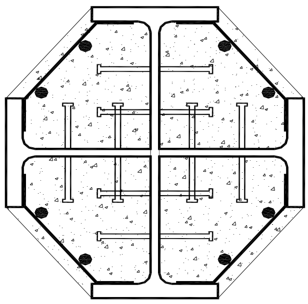


Fig. 3 Structural system



(a)



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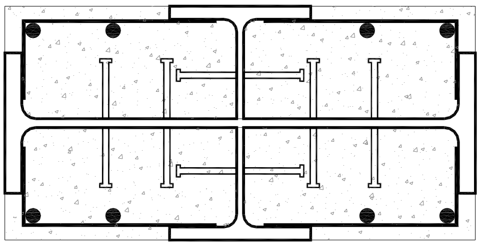

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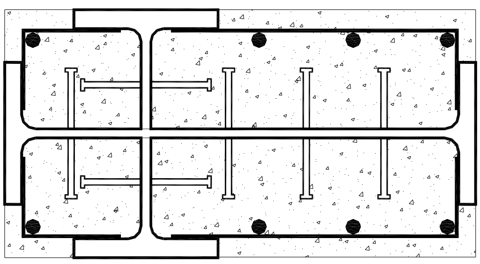


(d)

Fig. 4 Composite cross-sections

Each underground floor has 2360 m^2 and the three upper technical levels have 169 m^2 each. The entire surface of all floors is of around 30.000 m^2, and the total height of the building is of 106.3m

Fig. 1 shows the facades and two sections of the building while a photo of the building during construction is show in Fig. 2. The structure is made of composite steel and concrete columns and reinforced concrete floors connected to steel beams. Being located in a strong seismic area, the building structural system is heavily braced (Fig. 3).

The columns are made by partially concrete encased sections with crossed hot rolled European profiles. As Fig. 4 shows, there are four different cross section types for the columns:

- octagonal sections with identical steel profiles HEB500, HEA800, HEB800, and HE800 × 373 (Fig. 4(a));
- octagonal sections with different steel profiles HEM800-HEM700, HEB800-HEB700 and HEA800-HEA700 (Fig. 4(b));
- double-symmetric rectangular sections with different steel profiles HEB1000-HEB500 (Fig. 4(c));
- rectangular sections with one axis of symmetry, with different steel profiles HEB1000-HEM500 and HEB1000-HEB500 (Fig. 4(d)).

Longitudinal reinforcement with stirrups welded on the steel profile was provided, as shown in Fig. 4.

The beams are made of IPE450 to IPE550 sections, except for the beams connected to the inverted V braces, which are made of HEB800 sections. Headed stud connectors welded to the top flanges of the beams ensure the composite action of beams and concrete slab. In order to allow the development of plastic hinges in the moment resisting frames for the seismic action, no stud connectors were provided near primary beams ends.

- The centric X braces are made of hot rolled sections, varying from HEB450 in the lower storeys to HEA340 in the upper ones. Similarly, the inverted V braces vary from HEB450 to HEA340.
- The seismic design was performed according to the Romanian seismic code P100-1/ 2006. The design ground acceleration is a_g = 0.24 g, corresponding to a reference return period of 100 years. In order to obtain a favourable plastic mechanism, S355 steel was generally used for the frame members, excepting for the braces which use S235 steel, designed as dissipative members.

“Bucharest Tower Center” is a “very tall building” according the Romanian fire safety code P118-99 (1999), because the last floor is higher than 45 m from the fire cars access near the building. This classification imposes some specific conditions regarding the passive and active fire protection measures. The code provides five global fire safety levels, named “fire resistance degrees”, in which the main building elements are characterized by a minimal fire performance. For each fire resistance degree, there are limitations of maximal surfaces and number of floors. It is compulsory that all tall and very tall buildings must correspond to the minimal fire safety performances for the first degree. According to this classification, 150 minutes of fire resistance is required for the columns of “Bucharest Tower Center” building.

The building has four escape stairs. Two are disposed in cross ramps, separated by fire resistant elements. Other two stairs are placed at the opposite side of each office floor level. In order to avoid the fire spread on the facade, vertical curtain walls were provided with 1.20 m high compact screens.

Automatic fire alarm, linked to the devices for smoke exhaust in case of fire was provided for the entire building. The mechanical smoke exhaust for every underground floor was made separately.

The building is provided with interior hydrants, sprinklers, water curtains, exterior hydrants, dry vertical ducts in the staircases and manual extinguishers. Water curtains were provided to separate the car ramps in the underground car park.

Being a very tall building, "Bucharest Tower Center" is provided with its own fire brigade. The access of the fire brigade in the building is also possible through two special elevators (out of six) with two hours autonomy time in case of fire and through marked glazed on the exterior of facades.

3. Numerical fire analysis of composite columns

The columns of "Bucharest Tower Center" building, considered as isolated elements, loaded with the axial force and the bending moments on both principal cross-section axes (internal forces corresponding to the fire combination of actions), were modelled in SAFIR with 3D beam elements, taking into account that the internal moments have important values in both directions of the principal cross-section axes.

The analysis considered the following two load combinations of actions for accidental design situations, according to EN 1990 (2004)

$$\text{C1: } 1.00\,P + 0.50\,L$$
$$\text{C2: } 1.00\,P + 0.20\,W + 0.3\,L$$

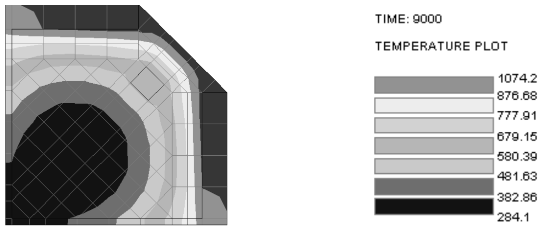
in which P is the dead load, L is the live load and W is the wind load.

The thermal action considered was the standard ISO fire, according to EN1991-1-2 (2005).

Equivalent imperfections, according to Table 6.5 of EN1994-1-1 (2005) for partially concrete encased sections with crossed steel profiles, were imposed on both directions of the principal cross-section axes. For both directions, the amplitude of the initial sinusoidal imperfection introduced in the numerical model was 1/200 of the length of the column. The buckling length of the columns was considered as the height of the corresponding storey.

Fig. 5 shows, for example, the results of the thermal analysis (temperature distribution) on the cross section of the octagonal column with identical steel profiles HEB500 (Fig. 5(a)) and on the rectangular section with one axis of symmetry, with different steel profiles HEB1000-HEM500 (Fig. 5(b)), after 150 minutes of ISO fire. Due to symmetry, only a quarter or half of the cross-sections is represented. The round reinforcing bars are represented by quadrilateral elements, with equivalent area. For all cross-sections, after 150 minutes of ISO fire, the steel profiles flanges exhausted practically their load capacity, having temperatures greater than 1000°C, while the webs of the steel profiles and the reinforcing bars have lower temperatures. The thermal analysis also emphasised an important core of concrete with quite low temperatures.

Consequently, after 150 minutes of ISO fire, the sections have a reserve of load capacity and the fire resistance demand may be fulfilled, function of the load level, given by the combination of actions in fire situation. The mechanical analysis under elevated temperatures was performed for each cross-section type, for the different floors of the building. The fire resistance grows with each floor, as the stress level in the columns with identical cross-section decreases on the height of the building.



(a)

(b)

Fig. 5 Thermal analysis for 150 minutes of ISO fire

Table 1 Minimum fire resistance times [minutes]

Column cross-section	Ground floor	Floors 1-10	Floor 11 - forth
Octagonal with identical steel profiles (excepting for HEB500)	> 150	> 150	> 150
Octagonal with identical steel profiles HEB500	70	100-149	> 150
Octagonal with different steel profiles	146	> 150	> 150
Double-symmetric rectangular	143	> 150	> 150
Rectangular with one axis of symmetry	> 150	> 150	> 150

At the ground floor level, the columns with rectangular cross-section with one axis of symmetry and the octagonal columns with identical steel profiles (excepting for the cross-sections with HEB500 profiles), fulfil the fire resistance demand. All the other columns of the ground floor do not resist up to the 150 minutes of ISO fire, under the imposed static loads. From the first floor and above, all columns fulfil the 150 minutes fire resistance demand, except for the columns with octagonal sections and identical steel profiles HEB500. For these columns, the fire resistance time of 150 minutes is reached only from the 11th floor forth.

Table 1 summarises the fire resistance times for the different types of cross-sections of the composite columns, as resulted from the numerical analysis under ISO fire. All the columns on the ground floor that do not fulfil the fire resistance demand need to be fire protected, while the columns with octagonal sections and identical steel profiles HEB500 need fire protection up to the 11th floor.

A numerical analysis of the columns was also performed by the authors under the action of natural fire (Zaharia *et al*. 2007), considering the particularities of the building, including a variation of the openings, taking into account the glass breaking function of the temperature, based on available experimental results. Considering a natural fire scenario, all the columns of

"Bucharest Tower Center" building fulfil the fire resistance time of 150 minutes, at all floors, but this analysis was not considered for the fire design, and adequate protection of the columns was finally recommended, based on the analysis under ISO fire.

4. Validation of the advanced calculation model

4.1 Presentation of the test

ARBED Luxembourg has promoted partially concrete encased sections with crossed steel profiles, in order to build strong composite steel-concrete columns with high fire resistances, without supplementary fire protection. For the validation of the advanced model SAFIR, requested by the Romanian authorities, one test on such a composite column, with cross section similar to the columns of the "Bucharest Tower Center" building, was selected from the experimental report REFAO (1987). The experimental research summarised in this report consisted of fifteen full-scale ISO fire tests of composite beams and columns, performed by Arbed Luxembourg since 1982 up to 1985 and sponsored by C.E.C., the Commission of the European Community.

Fig. 6 shows the experimental set-up of the composite column considered for the validation.

The column was composed of three hot-rolled H profiles, welded together and concreted between the flanges. Two HEA180 profiles were welded on the web of an IPE400 profile, as Fig. 6 shows. The composite section did not contain any supplementary longitudinal reinforcing bars. A steel mesh was provided at the outer concrete parts between the steel flanges, as shown in Fig. 6, in a similar manner as the welded stirrups on the steel profile flanges provided in case of the columns of "Bucharest Tower Center".

The characteristics of concrete were determined on cubic samples of 200 mm, tested before the fire test. The obtained values for resistance were used for the computation of the resistance on cylindrical samples of 150mm diameter and 300 mm height, in accordance with C.E.B. (1964), thus resulting a compressive strength of 51.90 N/mm^2. In order to prevent the spalling of concrete during the fire test, all composite test elements were concreted four to five months in advance. The age of the concrete at the test date, for the column in Fig. 6 was of 168 days. The water content of concrete for the tested specimen was evaluated to 80 l/m^3, but no moisture was considered in the numerical analysis.

The yield strength of steel of 299.90 N/mm^2 was determined on samples taken from the profiles used in the fire test.

The tested column had no measurable initial imperfections on its length and was loaded with a compressive force of 1600 kN, with a zero eccentricity. However, the numerical simulation considered a small initial sinusoidal imperfection of the element, with a maximum value of 1 mm.

The octagonal composite column behaved quite well in the fire test, as the resistance time attained 172 minutes, though four visible unprotected steel flanges were exposed directly to the fire action. Few spalling occurred, so that the concrete mass remained intact up to the failure of the column. This was due to the presence of the steel mesh placed between the steel profiles flanges (REFAO 1987).

4.2 Thermal analysis validation

In a numerical thermal analysis, the heat transfer through convection and radiation from the gas to the section of the structural element is given by the net heat flux, as presented in EN 1991-1-2

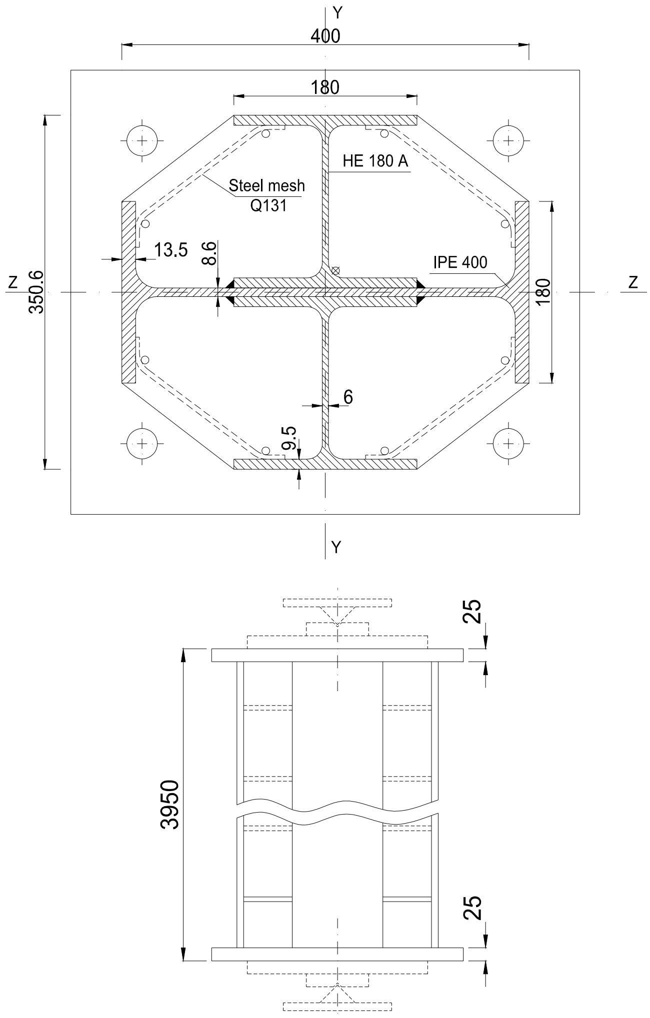


Fig. 6 Composite column (REFAO 1987)

(2005). The net radiative heat flux depends on the product between the configuration factor and the resultant emissivity.

According to Annex G of EN 1991-1-2 (2005), the configuration factor Φ measures the fraction of the total radiative heat leaving a given radiating surface that arrives at a given receiving surface. Its value depends on the size of the radiating surface, on the distance from the radiating surface to the receiving surface and on their relative orientation. In fire design, unless proper calculations are made to consider the position and the shadow effects, the configuration factor may be conservatively considered with the unit value.

The resultant emissivity is given by the emissivity of the fire ε_f multiplied with the surface emissivity of the member ε_m. The emissivity of fire may be conservatively taken with the unit value, as EN 1991-1-2 states. The emissivity of the member depends on the material. For both carbon steel and concrete, EN 1994-1-2 gives the surface emissivity $\varepsilon_m = 0.70$.

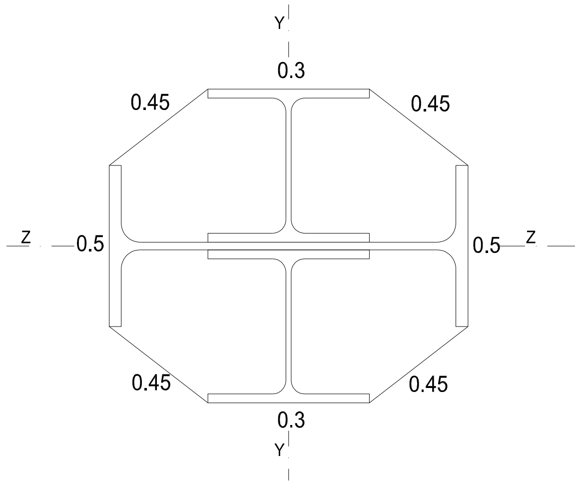


Fig. 7 Resultant emissivity for profile flanges and for concrete (REFAO 1987)

In a fire test, the radiative flux depends on the relative position of the exposed surfaces to the burners, the dimensions of the furnace and its characteristics. For the furnace of Gent University in Belgium, where the octagonal composite column was tested, the values of the resultant emissivity considered for the surfaces of steel elements (0.30 and 0.50), and for the concrete (0.45), are given in Fig. 7 (REFAO 1987). These values, which include the effect of the configuration factor, were considered in the numerical analysis.

Fig. 8 shows the comparison between the measured temperatures within the composite cross section of the column (where the thermocouples were placed) and the temperatures obtained from the numerical analysis under ISO fire, in the same points of the cross-section, at the failure time of the tested column of 172 minutes (Zaharia and Dubina 2012). Fig. 9 shows the distribution of the temperature on the cross-section, at the same time of 172 minutes, considering for the entire perimeter of the cross-section the value of the product between the configuration factor and the resultant emissivity as 0.70, as given in EN 1994-1-2 (2005).

The numerical thermal analysis offers good results, with results closed each other, in the safe side (higher temperatures), if the measured values of the resultant emissivities are considered. If the EN 1994-1-2 (2005) values for these parameters are considered, the results are more conservative.

4.3 Mechanical analysis validation

According to the clauses of EN 1994-1-2 (2005) regarding the validation of the advanced calculation models, for this particular case, a sensitivity analysis was also performed.

The sensitivity analysis conducted within the mechanical analysis under elevated temperatures, considered the following critical parameters: the buckling length, the initial imperfections, the level of the load and the resultant emissivities for steel and concrete. Table 2 summarises the results of this analysis, which demonstrates that the advanced model SAFIR complies with the sound engineering principles: the fire resistance time increases with the decrease of the buckling length of the column, of the initial imperfection, of the load level and of the resultant emissivity (which leads to lower temperatures on the cross-section). The level of the load of 100% in Table 2 represents the full compressive load of 1600 kN, applied on the tested specimen. Of course, the program SAFIR already demonstrated its capabilities from this point of view. The results presented herein were given for the validation purpose, which was approved in this form by the Romanian authorities.

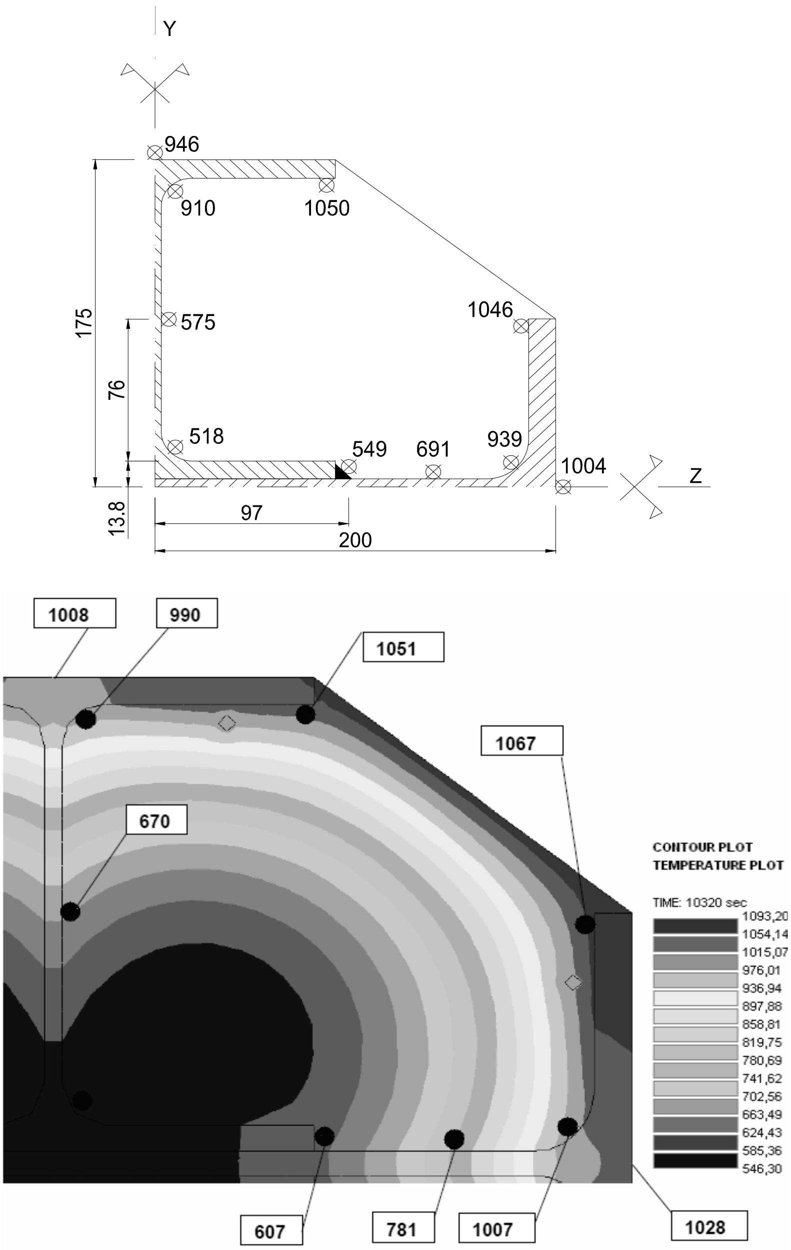


Fig. 8 Distribution of temperature on cross-section – experimental (REFAO 1987) and numerical, considering test values of emissivities

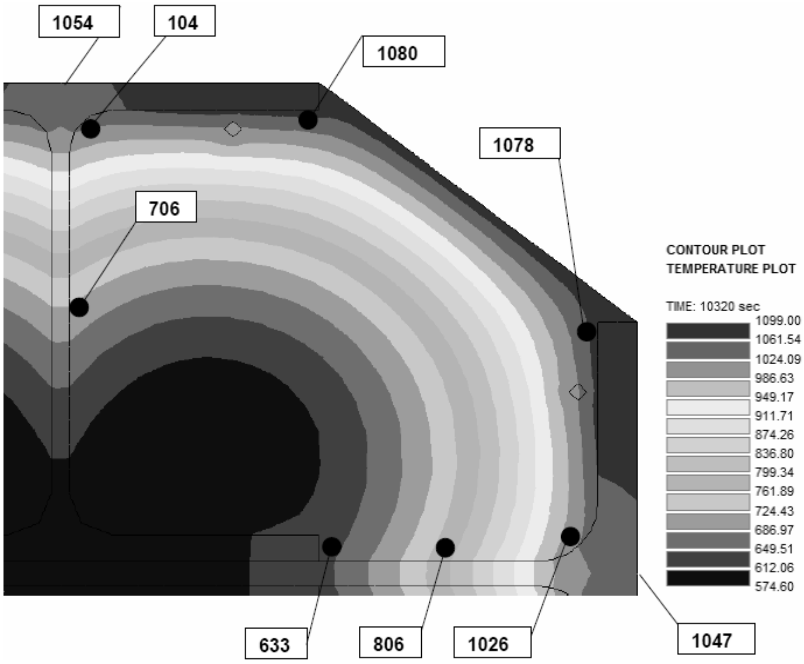


Fig. 10 Distribution of temperature on cross-section – considering Eurocode values of emissivities

Table 2 Sensitivity analysis

Case	Parameters				
	Buckling length	Resultant emissivity	Initial imperfection	Load	Fire resistance [min]
1	1.00 L	test	1 mm	100%	132
2	0.50 L	test	1 mm	100%	188
3	0.70 L	test	1 mm	100%	164
4	0.70 L	test	1 mm	75%	190
5	0.70 L	test	1 mm	125%	144
6	0.70 L	test	1/1000 L	100%	156
7	0.70 L	test	1/200 L	100%	140
8	0.70 L	EN 1994-1-2	1 mm	100%	152
9	0.70 L	EN 1994-1-2	1/1000 L	100%	147
10	0.70 L	EN 1994-1-2	1/200 L	100%	128

Some remarks must be made concerning the results presented in Table 2. Considering the buckling length equal to the length of the column (both ends are pinned, as supposed in the experiment), the fire resistance time given by the numerical analysis is of 132 minutes (case 1 in Table 2, in which the resultant emissivities from test and a small initial imperfection of 1 mm were considered). This is a conservative result, in comparison with the fire resistance time of 172 minutes reached by the experimental specimen.

If a buckling length taken as half of the column length is considered (corresponding to the situation in which both ends are fixed), while all the other parameters remain the same in the numerical analysis (case 2 in Table 2), the fire resistance time is of 188 minutes, higher than the fire resistance time obtained in the test. It is obvious that, with the connection details of Fig. 1, no fixed ends can be assumed, and therefore this could not be a realistic assumption.

If a buckling length of 70% from column length is considered (classic intermediate situation between fixed and pinned supports), the fire resistance time given by the numerical analysis is of 164 minutes (case 3 in Table 2). This suggests that for the tested column it was not possible to realize perfect pinned ends, and a certain degree of restraint of both ends of column was present. It must be mentioned that the structural element is a slender element (taking into account its length and the dimensions of the cross-section), and therefore, even for a reduced degree of restraint at its ends, the results of the numerical analysis might be affected. Obviously, it is practically impossible to evaluate the level of the rotational restraint at the ends of fire-tested member; therefore, the intermediate situation between perfect pinned and perfect fixed was further considered for the sensitivity analysis (cases 4-10 in Table 2).

Regarding the imperfections, even if the initial geometrical imperfection of 1/1000 L is not included in the recommendations of EN 1994-1-2 for the advanced calculation of isolated members (as it is in EN 1993-1-2 (2005)), it was however considered in the analysis. The results demonstrate that an initial imperfection of 1/200, as recommended in EN 1994-1-1 (2005) for these types of composite cross-sections for calculations at ambient temperature, and as used in the fire resistance verification of the columns of “Bucharest Tower Center” building, is a conservative assumption.

5. Conclusions

The computer program SAFIR for the analysis of structures in fire conditions is a worldwide-utilized code for structural fire analysis and has already proved its capabilities. For the numerical assessment of the fire resistance of the composite columns of “Bucharest Tower Center” building, a supplementary validation of the advanced calculation model SAFIR was performed at the demand of the Romanian authorities, considering the results from a fire test on a similar composite steel-concrete column.

Compared to the test, SAFIR gives good results, in the safe side, for both thermal and mechanical analysis, if all the parameters measured experimentally are considered. If the Eurocode specifications for the initial imperfections and resultant emissivities (as used for the calculation of the fire resistance of the columns of “Bucharest Tower Center” building), are introduced in the numerical analysis, more conservative results are obtained. As expected, the sensitivity analysis showed that the computer program SAFIR offers appropriate results, in accordance with the engineering principles. Following this procedure of validation, the advanced calculation model SAFIR was accepted as a reliable tool to evaluate the fire resistance of the composite columns of “Bucharest Tower Center” building.

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