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Recent developments and fire design provisions for CFST columns and slim-floor beams

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ABSTRACT

This paper summarizes the latest technical and scientific progresses on steel-concrete composite structures exposed to fire, presenting the recent research carried out on this subject and the progress of the design codes. In particular, this review focuses on concrete-filled steel tubular columns and slim-floor beams, topics where the authors have carried out extensive research during the last years. The more recent experimental and numerical studies performed by the authors as well as those available in the literature are presented, along with applications where these composite elements have been used in practice. The use of advanced materials, such as high strength steel and concrete, stainless steel, lightweight concrete or geopolymer concrete is considered for the enhancement of the fire behaviour of concrete-filled steel tubular columns and slim-floor beams. Finally, the currently available design methods for the calculation of isolated members at elevated temperatures are reviewed and the recent progress of the code provisions for the fire design of these composite elements is presented.

Keywords: *Concrete-filled steel tubular columns, slim-floor beams, fire resistance, innovative sections, advanced materials, design codes*

1. INTRODUCTION

Steel-concrete composite construction has driven the attention of researchers during the last decades, leading to important developments in this field, what in turn has provided practitioners with new solutions and techniques for taking advantage of the combination of steel and concrete in new buildings.

Amongst the main advantages of composite construction, special mention should be done to their enhanced fire performance, owing to the heat sink effect provided by the concrete, which delays the temperature rise in composite sections as compared with bare steel solutions.

This paper focuses on the fire performance of steel-concrete composite structures, and is divided into two main parts: composite columns and composite beams, both of which have been studied by the authors through numerical models and extensive experimental testing. The recent developments and current trends in the use of composite solutions are reviewed in each part, as well as the current design provisions and available calculation methods. Although different configurations for composite columns and beams exist in the market (i.e. fully encased, partially encased, non-encased, etc.), only certain types of composite solutions will be studied in depth in this paper, those in which the authors have focused their research during the last years, being at the same time one of the most frequently used solutions for composite construction in practice. The first part of the paper is focused in particular on concrete-filled steel tubular (CFST) columns, while the second part of the paper specifically addresses a novel type of composite beams: the so-called slim-floor beam.

It must be mentioned in this point that the behaviour of an isolated member is different to that of the same member within the complete structure, therefore the recommendations given in this paper will be applicable to individual members, while the fire performance of the whole composite system should be evaluated through a global model that accounts for relevant aspects such as the stiffness of the connections, axial and rotational restraints, membrane action, fire exposure conditions, etc.

2. STEEL-CONCRETE COMPOSITE COLUMNS

Steel-concrete composite columns are columns that combine the advantages of steel and concrete working together. Totally or partially encased composite columns, concrete-filled steel tubular columns or composite columns with inner steel profiles can be found in practice, see Fig. 1. One of the most commonly used composite column types are concrete-filled steel tubular columns, which have been the focus of research of the authors during the last years. The following section summarizes the current state of the art on this topic, as well as the latest research carried out by the authors on this field.

2.1. Concrete-filled steel tubular columns

The use of concrete-filled steel tubes (CFST) has increased in recent decades, finding an important demand in the construction of high-rise buildings, bridges and offshore structures, owing to their high load-bearing capacity, high ductility and energy absorption ability [1]. Other applications where this typology can be found are industrial buildings, electricity transmitting poles, subways, open car parks, office or residential blocks [2].

Circular, square, elliptical and rectangular steel tubes are most commonly used to form these composite columns (see Fig. 2), although new shapes are emerging in the market, such as polygonal or round-ended sections.

CFST columns show an excellent structural performance, taking advantage of the combined effect of steel and concrete working together - the steel tube provides confinement to the concrete core, resulting in increased compressive strength, while the concrete core restricts inward deformation of the steel tube thus enhancing local buckling resistance and enabling the use of thinner cross-sections -.

Even though the steel tubes can sometimes be filled with plain concrete, in most cases the concrete infill is reinforced with steel bars or with metal fibres. The presence of rebars not only decreases the propagation of cracks and sudden loss of strength, but also contributes to the load-carrying capacity of the concrete core [3]. However, in some situations, this solution is not feasible, as the placement of rebar caging inside the steel tubes is obstructed by the shear studs and/or by the inner diaphragm plates at the junction of beam to column joints. As an alternative, the use of fibre-reinforced concrete filling often results in fire resistance values which are comparable to those of CFST columns with reinforcing bars. These solutions may help avoiding the use of external fire protection for a certain range of fire resistance requirement.

2.1.1. Applications

Numerous examples of composite structures with circular or square CFST columns, as external or internal structural members, can be mentioned.

In China, concrete-filled steel tubes have been employed in construction since 50 years ago, being used as the main compression resisting components [4]. A good example is the Canton Tower in Guangzhou (see Fig. 3a), comprising twenty-four inclined circular CFST members with a maximum diameter of 2000 mm and wall thickness of 50 mm.

Further examples can be found in Northern America [5]. In the Museum of Flight at King County Airport (Seattle, Washington, USA) bar-reinforced concrete filled hollow sections are used for the columns supporting the roof of the exhibit hall, allowing to fulfil the required fire resistance without the need of sprayed fire protection. Another application can be found in the St. Thomas Elementary School (Hamilton, Ontario, Canada), where concrete filled CHS columns with different concrete strengths are used, achieving one hour fire resistance rating.

In Australia, the Riverside Office Building can be cited [6], using concrete filled bar-reinforced CHS columns of 600 mm diameter which were required to fulfil a 120-minute fire resistance. The Commonwealth Centre in Melbourne, the Forrest Centre, Exchange Plaza and Westralia Square in Perth are additional examples of such type of construction in Australia.

Several examples can also be mentioned in Europe, mostly in the United Kingdom [7], such as NEO Bankside, the Fleet Place or the Rochdale bus station. The Peckham Library is another example of such type of construction in London. Seven external circular CFST columns support the building at the front (see Fig. 3b). The inclined 18 m long columns meet the 60-minute fire rating required without any external protection.

In Marguerite Yourcenar Media Library (Fig. 3c), circular CFST columns are utilised, typically 273×10 mm S355 steel tubes, filled with C40 to C50 concrete and 16 mm S500 steel bars. The 3.90 m long inclined columns achieve the 120-minute fire resistance required. The Tecnocent Building (Oulu, Finland) makes use of circular and square bar-reinforced concrete filled hollow section columns [6]. The Mjärdevi Centre in Sweden is a twelve-storey office building with 200 mm diameter circular CFST columns, continuous over 3 storeys. ArcelorMittal Steel Centre in Liege, Belgium is a five-storey office building comprising external unprotected circular columns. In Germany, the City Gate in Düsseldorf is a high-rise building composed of two sixteen-storey towers connected by a 3-storey attic to a portal. Circular CFST columns are used, in combination with concrete partially encased beams (R90).

Concrete-filled steel tubular members have also been applied in many types of bridges [1], such as arch bridges, cable stayed bridges, suspension bridges, and truss bridges. CFST members can serve as piers, bridge towers and arches. An important advantage of using CFST in an arch bridge is that, during the stage of erection, the hollow steel tubes can serve as the formwork for casting the concrete, which significantly reduces the construction cost.

Although the described construction examples give an idea of the good fire performance of CFST columns, in applications which require a high slenderness combined with important bending moments, the magnification of the second order effects question their applicability.

2.1.2. Experimental investigations

A great number of fire tests on CFST columns have been performed from the 1970s to the 2000s in Europe (France [8, 9], UK [10], Germany [11] and Spain [12, 13]), in Northern America (Canada [14]) and in Asia (China [15]), which have contributed to make available to the research community a wide database of experimental results. A compendium of these test results can be seen in [16]. For circular and square columns, the size of the steel tube ranges from 141.3×6.55 to 478×8, with a length of the column from 3.18 m to 5.2 m. In general, the columns were fixed at least at their bottom end. While most of the experimental tests available in the literature have focused on circular or square concrete-filled steel tubular columns, the number of available experimental results on concrete-filled rectangular columns is reduced, being the main contributions to be cited those from Han and co-workers [17]. A total of 8 concrete-filled rectangular hollow section columns were tested under axial or eccentric loads. Two different sections were used: 300×200×7.96 mm and 300×150×7.96 mm. The effect of the fire protection thickness was investigated: four column specimens were unprotected, while the other four were externally protected with spraying coat.

The available experimental investigations on concrete-filled elliptical columns at elevated temperatures are even more limited: six tests of cross-sectional dimensions 220×110×12 mm carried out by the authors [18] and 18 tests carried out by Ali et al. [19] on hollow and concrete-filled elliptical sections subjected to hydrocarbon fire, with sectional dimensions 200×100×8mm, 300×150×8 mm and 400×200×8 mm.

In order to extend the available test database on circular and square CFST columns [20] and cover the lack of experimental evidence on rectangular and elliptical sections [21], an extensive fire testing programme was carried out by the authors in the framework of a project funded by the Research Fund for Coal and Steel (RFCS) of the European Commission – FRISCC “Fire Resistance of Innovative and Slender Concrete Filled Tubular Composite Columns” – [16] consisting of 36 fire tests, where CFST columns with circular, square, rectangular and elliptical cross-section combining high slenderness and large eccentricities were tested. The studied parameters were the cross-section shape, sectional dimensions, member slenderness, load eccentricity and reinforcement ratio. An example of one of the columns before and after test can be seen in Fig. 4.

From the results of these fire tests, it was found that, for the same steel usage, the circular columns presented a better fire performance than the square columns. Additionally, for the same column dimensions and percentage of reinforcement, the fire resistance time was significantly reduced when introducing eccentricity. Furthermore, it was found that for the same load eccentricity, when the percentage of reinforcement was increased, the fire resistance time also increased.

The results of this experimental campaign also highlighted the limited fire resistance of CFST columns with high slenderness. The premature failure in slender columns was found to be due to a local behaviour which occurs close to the column ends, an issue which must be solved for optimizing their fire response. Therefore, innovative solutions are needed which help improving the performance of this typology of composite columns in the fire situation.

2.1.3. Load-bearing behaviour in fire and failure mechanism

The typical behaviour of a CFST column subjected to a standard fire test can be divided into four stages, which can be clearly distinguished in Fig. 5. This figure shows the evolution of the axial displacement-time curve together with the axial force ratio versus time curves for both the steel tube and concrete core during the full-range fire exposure.

As it can be seen, at the start of the fire test, the steel tube heats up more rapidly and expands faster than the concrete core (stage 1), since it is directly exposed to the heating source. The higher thermal conductivity of steel accelerates the heating of the outer tube and thus its thermal expansion. Because of this faster axial elongation of the steel tube and the occurrence of slip at the steel-concrete interface, the concrete core loses contact with the loading plate, thus the axial load ratio of steel progressively increases until the whole applied load is sustained by the steel tube alone. The outer tube remains fully loaded during a significant period of time until the critical temperature of steel is reached. At this point, the local yielding of the steel tube occurs and it starts to shorten (stage 2), allowing for the loading plate to contact the concrete core again. As the column shortens, the steel tube progressively transfers the load to the concrete core (stage 3) and an inversion in the axial force ratio takes place, in such a way that the concrete core becomes the main resistant element of the column, since the steel tube has already lost its load-bearing capacity. Because of its low thermal conductivity, the concrete core degrades slowly as temperature advances through the cross-section, until eventually the column fails when concrete loses completely its strength and stiffness (stage 4).

A more detailed analysis of the load transfer process, with an insight into the stress distribution within the cross-section, can be obtained from [12]. It was found that the cross-sectional temperature gradient within the concrete core gives rise to a field of self-balanced thermal stresses, which are compressive in the outer layers of concrete and tensile in the central part. The stresses in the central layers of the concrete core remain positive and increasing with temperature until the applied load is transferred from the steel tube to the concrete core, when the stresses in that central part of the concrete undergo an inversion, changing from tension to compression. Moreover, an ovalization of the section was observed, producing a certain amount of lateral pressure into the concrete core. However, this non-uniform lateral pressure distribution in case of fire was found not to have a significant influence over the compressive strength of concrete at elevated temperature, thus the confinement effect should be neglected in fire.

From this analysis it was recommended to secure a small ratio between the applied axial load and the hollow steel tube capacity so as to allow for a proper transference of load from the steel tube to the concrete core, thus taking advantage of the contribution of concrete to lengthen the fire resistance time of the column.

2.2. Innovative composite columns

2.2.1. Concrete-filled dual steel columns

Recently, innovative steel-concrete composite solutions have been developed, which can solve the current limitations of slender CFST members when exposed to fire. At the same time, as the construction of high-rise buildings increases worldwide, solutions which allow for higher capacities at room temperature are sought by designers. One of these solutions are the so-called concrete-filled double-skin tubes (Fig. 6a), which have the potential to be used as columns in high-rise buildings, bridge piers or transmission towers [2],[22]. In this tube-in-tube configuration, the inner steel tube is thermally protected by the outer concrete ring and therefore its degradation when exposed to fire is delayed, which may help resisting the applied load for a longer period of time, solving the aforementioned problems of slender CFST columns in the fire situation.

The fire performance of these columns can be enhanced even more by adding concrete inside the inner tube, constituting the so-called double-tube columns (Fig. 6b), where both the inner and outer tube are filled with concrete. Filling the inner steel tube with concrete contributes to increase the load-bearing capacity of the column, while it delays the temperature rise within the column cross-section and therefore lengthens its fire resistance. This solution can be found in practice, being a good example the Queensberry House in London (UK) [7], a six-storey office and commercial building where the columns use a tube-in-tube system in which one CHS section is placed inside a larger one with all the voids grouted after erection of the floor structure. No external fire protection was needed, as the internal composite column had enough load-bearing capacity by itself in the fire limit state.

These two types of sections, with or without filling the inner core with concrete, can be generally categorized as concrete-filled dual steel columns (CFDST). Up to now, the main work on CFDST columns has been performed by Profs. Han, Zhao, Tao and co-workers, where several papers [23-25] have been published at room temperature and fire about the so-called “double-skin” columns, with the inner steel tube unfilled. In turn, the work by Liew et al. [26] has been focused on the so-called “double-tube” cross-sections, where the inner tube is also infilled with concrete.

These investigations confirmed that CFDST columns perform better at elevated temperatures than unfilled and conventional concrete-filled steel tubular columns. In particular, Lu et al. [27] carried out six fire tests on self-consolidating concrete (SCC) filled double-skin tubular columns (CFDST) with circular and square cross-section without concrete at the inner tube, where the so-called “cavity ratio” was found to affect the fire endurance of the composite columns.

In a more recent investigation [28], the authors of this paper presented the results of an extensive experimental campaign carried out at the Polytechnic University of Valencia (Spain) consisting of 30 concrete-filled dual steel tube column tests, where the effects of two parameters was analysed: strength of concrete (normal strength and ultra-high strength concrete) and the ratio between the thicknesses of the steel tubes. From these tests, six of them were performed under fire conditions. Four of the column specimens were filled with concrete at the inner core (normal or ultra-high strength concrete, i.e. double-tube), while the other two columns were only filled at the outer concrete ring (i.e. double-skin). Normal (C30) and ultra-high strength (C150) concrete were used for filling the columns. The influence of filling the inner tube with concrete (i.e. double-tube section) was studied, as well as the variation of thicknesses of the outer and inner steel tubes. It was found that a good design strategy for CFST columns in case of fire is to split the outer tube into two different steel tubes (outer + inner) with the same total steel area (and thus same steel usage), placing the thinner tube at the outer part of the section and the thicker tube at the inner part, so as to be thermally protected by the concrete ring (see Fig. 7). Moreover, it was recommended that both rings are filled up with concrete for an enhanced fire performance. However, this may be detrimental for room temperature design, as in that case it results optimal that the thicker steel tube is located at the outer part of the section for increasing the second moment of area and, in the case of stub columns, taking advantage of the enhanced confinement effect. Therefore, it is difficult to find a unique solution that simultaneously maximizes both the room temperature and fire load-bearing capacity.

2.2.2. Concrete-filled steel columns with embedded steel profiles

An alternative and innovative solution consists in embedding a steel profile within the concrete-filled steel tubular section. An example of the use of this solution is the Millennium Tower in Wien, Austria (see Fig. 8a). It is a fifty-five storey, 202 m high building, with external and internal CFST columns. These columns are made of outer circular S355 steel tubes with a

C40 to C60 concrete infill. In order to increase their load-bearing capacity, the internal columns were provided with embedded H profiles.

In the Netherlands, the Amsterdam Mees Lease Building is a four-storey office building using 323 mm CHS columns with a fire resistance rating of 60 minutes in combination with concrete encased HEA beams [6].

Amongst the scarce investigations that can be found on CFST columns with embedded inner steel profiles, the work from Dotreppe et al. [29] carried out at the University of Liege (Belgium) can be cited. In this research, four columns filled with self-compacting concrete (SCC) embedding an HEB profile were tested at elevated temperature.

2.2.3. Concrete-filled steel columns with massive embedded steel core

Other types of innovative solutions have recently emerged for optimising the cross-section of composite columns. Those columns consist of a hollow steel section, a massive embedded steel core and concrete infill in between. This cross-section type comes along with a significantly increased load bearing capacity compared to other types with identical outer dimensions. In consequence, either higher loads can be applied or columns can be designed with smaller dimensions. Neuenschwander et al. [30] or Schaumann and Kleibömer [31] have studied these type of sections. However, further experimental testing on this type of innovative composite sections is needed, in order to establish accurately their fire performance and determine the evolution of shear stresses at the steel-concrete interface.

This innovative cross-section has been recently used in projects of high-rise buildings. Two engineering companies in Germany can be cited, which have developed patents using this new type of cross-section: Spannverbund GmbH® and Stahl+Verbundbau GmbH®.

The Highlight Towers in Munich, finished in 2004 (see Fig. 8b), are one example of the use of innovative CFST columns with embedded steel cores, meeting a fire resistance class

R120. 750 out of a total number of 1400 columns have a massive steel core encased in concrete with an outer hollow steel profile. The embedded steel cores range from simple circular cross-sections to stepwise welded massive steel plates (see Fig. 8b).

2.2.4. Comparative study of the fire performance of different innovative solutions

The fire performance of these innovative solutions has been studied through a numerical model by the authors [32]. Fig. 9 compares the fire performance of several innovative steel-concrete composite columns, against a reference CFST column with a fire resistance of 28 minutes (a) tested in a previous experimental campaign. This CFST section of dimensions 273×12.5 mm is chosen as a reference, and it is used to generate other four innovative sections with inner profiles, maintaining the total amount of steel (same steel contribution ratio). As it can be seen, the embedded steel core solution (e) lengthens the failure time slightly, up to 36 minutes. In turn, with the embedded HEB solution (d), the fire resistance of the column is increased up to 47 minutes. Finally, if the steel tube is split into two tubes, generating the CFDST column with the thicker tube in the inner part of the section, the fire resistance is significantly improved to 77 minutes (b), provided that the inner tube is infilled with concrete (i.e. double-tube). In the case of not using concrete inside the inner tube (i.e. double-skin), there is still an improvement in terms of fire resistance to 63 minutes (c), although not as significant as that obtained with the double-tube solution.

2.3. Advanced materials

The previously exposed advantages of composite structures can be exploited with an efficient use of advanced materials emerging in construction, such as high strength steel (HSS), stainless steel (SS), high strength concrete (HSC) or geopolymer concrete (GPC). These materials may enhance the fire performance of composite steel-concrete solutions, depending on the part of the section where the advanced material is applied. Moreover, a more rational and efficient use of the material may lead to important savings, as a higher load-bearing capacity allows for section reduction. The addition of steel fibres to the concrete mass is considered as an alternative to the use of reinforcing bars for providing a suitable response at elevated temperature in applications where it is not feasible to place a rebar caging at the inner core. Also the introduction of environmentally friendly concrete types such as recycled aggregate concrete (RAC) is evaluated in this section, as a way to decrease the associated carbon footprint in composite construction.

2.3.1. High strength steel

In structural steelwork, high strength steels enable less material to be used, which reduces the costs associated with construction, transport and assembly. However, regarding their behaviour at elevated temperature, little information exists in the literature and the building codes do not include design recommendations for this type of steels in the fire situation. EN1993-1-12, related to HSS up to S700 grade does not provide any additional information on the fire design of such steel grades, and practitioners are referred to EN1993-1-2, valid up to S460 grade. Several investigations have focused on establishing the fire behaviour of HSS. Amongst the existing work, results from Lange and Wohlfeil [33], Schneider and Lange [34] and Outinen [35] on HSS S460, or Chen et al. [36] and Chiew et al. [37] for HSS S690 can be found. Recently, Qiang et al. [38], investigated the properties at elevated temperatures of HSS S460, S690 and S960, proposing reduction coefficients of the mechanical properties of these steels at elevated temperature based on experimental results. Other recent investigations by Choi et al. [39] have focused on the thermal and mechanical properties of HSS at elevated temperatures.

Recent studies by Xiong and Liew [40, 41] highlighted that HSS manufactured from different heat-treatment processes - i.e. thermo-mechanically controlled process (TMCP) and quenching and tempering (QT) process - present different mechanical behaviours under fire. In base of the coupon test results, heat-treatment dependent thermal elongations, elastic moduli and effective yield strengths at elevated temperatures were recommended by Xiong and Liew for HSS produced under different heat-treatment methods.

The improved mechanical properties of high strength steels opens a new range of possibilities regarding their application in CFST columns, where they can improve the problem of the limited fire resistance of slender members. In fact, Tondini et al. [42] presented recently the results of a fire test on a CFST column using HSS, where the superior performance of composite columns made of HSS was proved.

Singular buildings of recent construction have used HSS, such as the “Freedom Tower” in New York (USA), the Olympic Stadium “Bird’s Nest” in Beijing (China) or the Millau viaduct (France). In Japan, a new building in Kiyose City uses CFST columns combining 700 MPa HSS with ultra-high strength concrete. Liew and Xiong [43] present several examples of buildings using HSS in combination with HSC.

Taking advantage of the improved properties of HSS and using the appropriate steel share between the outer tubes and inner profiles, it may be possible to obtain an elevated fire resistance without the need for external protection, which will lead to subsequent cost and time savings.

Previous investigations performed by Espinós et al. [32] confirmed that a good strategy for enhancing the fire resistance of these composite columns is to improve the steel grade of the inner profile without reducing the total steel area (see Fig. 10). As it can be seen in this figure, in the case of the CFST-HEB solution, the fire resistance time increases from 47 to 86 minutes by using an inner profile made of S960. In turn, for the CFDST solution, a noticeable increase from 77 to 141 minutes is obtained. It should be noted that, apart from increasing the fire resistance time of the columns, their maximum capacity at room temperature is significantly increased by using a higher grade of steel at the inner profiles, while maintaining the outer dimensions. Therefore, by using HSS at the inner profiles both the load-bearing capacity of the columns at room temperature and their fire resistance can be enhanced at the same time. The use of inner steel profiles made of HSS offers an alternative to applying intumescent coatings, with better external appearance and no need for maintenance. However, the reduction of the degree of utilization for increasing steel grades should be taken into account in design.

2.3.2. Stainless steel

Other material that can be potentially used for enhancing the fire performance of composite columns is stainless steel [44]. Stainless steel tubular sections are becoming widely used in construction due to their beneficial characteristics such as corrosion resistance, high ductility, or good aesthetics. The reduction in strength and stiffness of stainless steel at elevated temperature is slower than for carbon steels [45], thus it can be exposed to higher temperatures without significantly altering its properties. A recent publication by Han et al. [46] reviewed the latest research on the topic and highlighted the benefits of the use of stainless steel in CFST columns. Authors such as Tao et al. [47], Han et al. [48] or Uy et al. [49] performed experimental tests on CFST columns with hollow tubes made of stainless steel, where their fire resistance was found to be much higher than those columns with outer carbon steel tube.

Compared to traditional carbon steels, stainless steel presents a high initial cost, which may be overcome by using the material more efficiently in innovative steel-concrete composite sections such as concrete-filled double-skin steel tubular (CFDST) columns, enhancing not only their room temperature capacity but also their fire resistance. Recent numerical [50] and experimental [51] investigations have been carried out on CFDST stub columns at room temperature with outer stainless steel tubes, where the improvement of strength and ductility attained by means of using this material at the outer tube was evidenced. However, the fire behaviour of steel-concrete composite columns with outer stainless steel tube and inner steel profiles has not yet been studied.

An initial comparison was carried out by the authors in order to evaluate the advantages of using this material at the outer tube in combination with inner steel profiles, as a continuation of the study presented in Espinós et al. [32]. The enhancement obtained by using an outer stainless steel tube is illustrated in Fig. 11, where the steel grade used at the inner profiles is S355. For the CFDST solution, the fire resistance time is increased from 47 to 62 minutes (31.91%), while for the CFST-HEB solution, an increment from 77 to 93 minutes is obtained (20.78%). The increment achieved with respect to the reference CFST column with carbon steel grade S355 is noticeable, using the same total steel area. Note that a fire resistance time higher than 90 minutes may be reached by combining an outer tube made of stainless steel with an inner tube of carbon steel grade S355 (93 min), leading to a higher fire resistance than that of the best combination of CFST-HEB in Fig. 10 (86 min). This solution may be an alternative to the use of HSS at the inner profiles, or may be used in combination with HSS for getting the best fire performance. This comparison should also be done in terms of material costs.

2.3.3. High strength concrete

Apart from taking advantage of advanced materials at the outer tubes or inner profiles, the load-bearing capacity of composite columns can be enhanced by increasing the strength of the concrete infill. High-strength concrete (HSC) has become an attractive alternative to conventional NSC in filling hollow section columns, due to the significant contribution that it can offer to sustain the applied load at room temperature. However, the benefits of using HSC at elevated temperatures are not so evident, since the behaviour of HSC at elevated temperatures is significantly different from that of NSC and this may have a significant effect on the resulting fire resistance of the composite columns. Kodur [52] studied experimentally the effects of concrete strength on the fire behaviour of CFST columns, finding that the fire resistance of HSC-filled columns was much less than that of equivalent NSC-filled columns subjected to the same load ratio. Therefore, solutions for enhancing the fire resistance of CFST columns filled with HSC should be used, such as steel-fibres or bar reinforcement. As the trend in room temperature design is to use increasingly higher strength materials for reaching higher capacities while reducing the sectional dimensions, a good solution may be found by combining HSC with reinforcement so as to maintain the competitiveness of HSC also at elevated temperature.

HSC may be also used as infill in double-skin or double-tube columns. Previous research by Romero et al. [28] on double-tube columns using ultra-high strength concrete (UHSC) up to 150 MPa revealed that filling the inner core with UHSC did not provide an enhancement in terms of fire resistance. In fact, the fire resistance time decreased as compared with an equivalent specimen filled with NSC, due to the faster deterioration of the mechanical properties of HSC.

Another important issue is the compatibility between high strength steel and concrete, which was addressed by Liew and Xiong [43]. In CFST columns subjected to axial compression, it should be ensured that the yielding of the outer steel tube occurs before the concrete core reaches its maximum stress, in order to achieve the full plastic resistance of the composite section. Thus, the steel grade and concrete class should be carefully selected in such a way that the yield strain of steel is smaller than the concrete strain at peak stress. For instance, steel grade S460 should be combined with concrete class C35 or higher, while S550 should be used in combination with concrete class C70 or higher. It should be noted that these recommendations only apply for room temperature design, whereas at elevated temperature the choice would not be trivial, as the temperature gradient progressively affects the steel and concrete strength from the outer surface inwards.

2.3.4. Fibre-reinforced concrete

Kodur and Latour [53] investigated the effect of adding fibres to the concrete mass in CFST columns, with the purpose of enhancing the fire performance of columns filled with HSC. It was found that the load-bearing capacity of HSC-filled columns was increased to a certain degree by adding the steel fibres, as compared to plain HSC-filled columns or bar reinforced HSC-filled columns, which was attributed to the fact that the compressive strength of fibre-reinforced concrete increases with temperature up to about 400ºC [54]. The steel fibres prevented early cracking and also contributed to enhance the compressive strength of concrete at elevated temperatures.

Based on the results obtained in several experimental campaigns carried out in Canada, Kodur [52] presented a number of broad guidelines for enhancing the fire resistance of HSC-filled hollow section columns. Kodur affirmed that by filling the columns with steel-fibre reinforced concrete, fire resistance ratings of up to 2 hours could be obtained even at load intensity of 0.7 (referred to the compressive resistance of the concrete core). As an alternative to steel fibre reinforcement, conventional bar reinforcement could be used to enhance the fire resistance. For high load levels, equivalent to the strength of the concrete core, Kodur recommended that rebars should be used. In such case, fire resistance times of up to 90 minutes could be obtained for columns with load intensity 1 (i.e. applied load equal to the compressive resistance of the concrete core) and up to 2 hours for load intensity of 0.75. It is worth noting, however, that these tests were performed under concentric load and that the slenderness of the columns was low to intermediate, producing in some cases the failure of the columns by concrete crushing instead of global buckling.

While Kodur [53] reported that steel fibres can be used to increase the fire resistance of high strength concrete-filled columns, the test results obtained in the tests performed by Romero et al. [12] did not seem to support this statement. The main difference between both experimental campaigns was that the columns tested by Romero et al. had a much higher slenderness. In these cases the steel fibres did not contribute to enhance the fire resistance of the column since the failure was mainly due to the hollow steel tube premature buckling before the load could be transferred to the concrete core. The steel fibres only contributed to enhance the fire resistance in those cases with a higher axial load level and thus higher influence of the second order effects, where the addition of fibres was more useful to increase the tensile strength of concrete. Therefore, the effectiveness of the use of steel fibres to improve the fire resistance in HSC-filled hollow section columns should be further evaluated depending on the slenderness of the column, being a possible alternative to the addition of reinforcing bars to achieve the required fire resistance.

2.3.5. Geopolymer concrete

The use of other types of novel concretes, such as geopolymer concrete (GPC) has also been considered in recent research. This type of concrete is an aluminosilicate binder with reduced associated CO_2 emissions and energy requirements, being a sustainable alternative to Portland cement concrete [55]. Geopolymer is produced by alkali activation of suitable aluminosilicate raw materials, such as metakaolin, fly ash or volcanic ash. While these materials are low cost, the prohibitive costs of laboratory grade activators greatly limit the widespread application of GPC [56]. This novel material has been so far used mainly in non-structural applications. In particular, it was used for making the pavements of the Westgate Freeway in Port Melbourne or the Brisbane West Wellcamp Airport in Australia. A structural application that can be cited is the case of the Global Change Institute building at the University of Queensland (Australia), where 33 floor beams were cast with GPC of grade 40 MPa.

Regarding its fire resistance, GPC presents a lower thermal conductivity and higher strength retention at elevated temperature than conventional concrete [57], thus resulting attractive for fire requirements. A previous investigation by the authors of this paper highlighted the potential of using the advantageous thermal properties of such type of concrete in composite columns for increasing their fire resistance [58].

Fig. 12 shows the results from a numerical study to evaluate the fire performance of double-tube composite columns using GPC at different locations within the section. It can be observed that, while the reference double-tube column filled with normal concrete attains a fire resistance time of 87 minutes, those columns using GPC at the ring between the outer and the inner steel tube lengthen their fire resistance time up to 139 minutes, experiencing a noticeable fire resistance improvement. This increase in fire resistance is due to the delay of the inner tube temperature rise provided by the outer ring of GPC. In turn, placing GPC at the inner core does not cause an improvement on the fire resistance of the column (in fact the fire resistance time is slightly lower than that of the reference column). Therefore, using geopolymer concrete in double-tube columns can provide an enhanced fire resistance provided that it is placed at the proper location within the section.

2.3.6. Recycled aggregate concrete

Recycled aggregate concrete (RAC) is an emerging sustainable type of concrete in which broken pieces of waste concrete are used as aggregate by cleaning, crushing and grading. The use of RAC in construction not only greatly reduces the demand for natural aggregate, but also reduces the environmental pollution caused by construction waste [59]. However, the mechanical properties of RAC have been found to be poorer than those of normal concrete with natural aggregates under the same mix proportions [60], reason why it is mostly used as non-structural concrete. The use of RAC in CFST columns, however, where it is confined and protected by an outer steel tube, prevents this material to be directly affected by harmful environment factors (i.e. temperature, water…), as it is the case of reinforced concrete structures.

The behaviour of recycled aggregate concrete-filled steel tubular (RACFST) columns at room temperature was experimentally studied by Yang and Han [61], where 30 columns including 24 RACFST columns and 6 normal CFST columns were tested. It was found that the RAC in-filled columns presented slightly lower but comparable ultimate capacities to their normal concrete-filled counterparts. The ultimate capacities of the circular columns filled with normal concrete were found to be 1.7%–9.1% higher than those of columns with RAC containing 25% recycled coarse aggregate and 50% recycled coarse aggregate, while for square specimens the differences ranged between 1.4%–13.5%.

The fire behaviour of RACFST columns was recently investigated by Liu et al. [59], where six specimens were subjected to axial compression tests after fire. It was found that the wall thickness of steel tube had a relatively large influence on the rate of loss of bearing capacity of RACFST columns after fire. The authors also confirmed that the load-bearing capacity of specimens with 100% replacement of aggregates was severely damaged in fire.

Yang and Hou [60] studied the behaviour of RACFST stub columns after fire exposure. Forty specimens, including 32 RACFST stub columns and 8 normal CFST stub columns for comparison were tested. Different replacement ratios (50% and 100%) were used for the RACFST specimens. It was found that the post-fire performance of RACFST stub columns was lower than the corresponding normal CFST specimens. Moreover, the ultimate strength and elastic modulus of RACFST specimens decreased with an increase of the recycled aggregate replacement ratio. Therefore, when using RAC in CFST columns, a well-balanced proportion of recycled aggregates and natural aggregates should be made, so as to take advantage of the reduced footprint of RAC without compromising the load-bearing capacity of a CFST column in fire.

3. STEEL-CONCRETE COMPOSITE BEAMS

Steel-concrete composite beams are built up by connecting a concrete slab to the top flange of a steel beam. This is normally done by shear connectors, which transfer the longitudinal shear at the interface between the concrete slab and the steel beam. The degree of shear connection influences the flexural capacity of the composite beam [1]. The mechanical properties of steel and concrete are best utilized in composite beams, since the concrete part is subjected to compression, while the steel parts are (partially) in tension.

Traditionally, steel-concrete composite beams have been built up by connecting composite, precast or in-situ concrete slabs on top of a solid steel beam (i.e. non-encased composite beam) or with a partially encased configuration, where the beam downstands under the floor, see Fig. 13. A novel type of composite steel-concrete beams fully embedded in floors, the so-called “slim-floors” emerged in the market at the beginning of the 1990s [62], characterized by incorporating the floor slabs within the depth of the steel beam (Fig. 14). The special arrangement of these types of beams makes it possible to place the slab elements directly onto the lower flange of the beam, resulting in an integrated and shallow solution. This configuration offers important advantages such as the floor thickness reduction, the increase of the working space, the ease for under-floor technical equipment installation and the disposition of a straight ceiling surface that makes the structure of the floor visible and facilitates the shifting of the inner partition walls. Because of these advantages, slim-floor beams are increasingly used in practice. This section focuses on this particular type of composite beam, which has been studied by the authors in the last years.

3.1. Slim-floor beams

As commented before, slim-floor beams are composite beams fully integrated into the floor depth. Different configurations of slim-floor beams have been made available in the construction market over the last decades, such as the "Thor-beam" [63] or the "Delta-beam" (Peikko®) [64] systems developed in Scandinavia, the "Asymmetric Slimflor Beams" (ASB) [65], the "Ultra Shallow Floor Beam" (USFB) [66] developed by Westok® in the UK, or more recently the so-called "Composite Slim-floor Beam" (CoSFB) developed by ArcelorMittal® [67], a special type of shallow floor beam with small web openings where the composite action between steel and concrete is ensured by means of transverse reinforcing bars through the beam web openings.

Attending to the configuration of the steel beam itself, two main types of slim-floor beams can be distinguished: Integrated Floor Beam (IFB, Fig. 15a) and Shallow Floor Beam (SFB, Fig. 15b). The former is made of a half I-section where a wider bottom plate is welded to the bottom of the web in replacement of the lower flange. The latter consists of a full I-section with a bottom plate attached and welded to its lower flange. Companies such as Profil-Arbed®, Stahl + Verbundbau GmbH®, Hoesch Siegerlandwerke GmbH® or British Steel® have developed slim-floor systems based on this type of beams (IFB or SFB) combined with either prestressed hollow core units, profiled steel decking with in-situ concrete or reinforced concrete slabs [68].

A suitable fire behaviour of these beams is expected, since the steel beam is totally embedded in the concrete floor and thus it results only exposed to fire from its lower flange. Additionally, the SFB configuration presents the advantage in fire of a thermal gap that appears at the interface between the steel profile lower flange and the bottom plate, which delays the temperature rise of the section, as observed experimentally by Newman [66]. Fellinger and Twilt [69] suggested that this thermal gap should be ensured in manufacturing the SFB specimens in order to increase the slim-floor fire resistance in practice.

Slim-floor beams can be used in combination with different flooring systems, such as in-situ concrete slabs, profiled steel decks or precast concrete slabs (Fig. 14). This last option provides additional benefits, such as its fast erection and structural efficiency for long spans.

Moreover, the slab configuration itself changes the incidence of the thermal action to the composite beam, i.e. the hot air between the ribs in a profiled steel deck facilitates the advance of temperatures in the beam section as compared to a floor configuration with concrete slabs.

3.1.1. Applications

Several examples can be found of buildings (particularly in Europe) where slim-floor beams have been used [68]. At the Ecole Nationale des Ponts et Chaussées (Marne-la-Vallée, France, 1994-1996) IFB elements composed of a half HP400 steel profile welded to a lower steel plate of dimensions 140x40 mm were combined with hollow core units plus an in-situ concrete layer of 8 cm on top to form a composite floor of 25 mm thick. Moreover, composite columns with partially encased steel profiles were used to form a full composite structure.

At the Profilarbed® office building (Esch-sur-Alzette, Luxembourg, 1991-1993) the floors were conceived with IFB beams composed of a half IPE500 welded to a lower steel plate of 420x10 mm, combined with hollow core units plus an in-situ reinforced concrete layer of 10 cm on top.

Although these are just a few examples of applications of slim-floor beams in practice, the previously mentioned advantages related to the floor thickness reduction and improved fire resistance are contributing to extend the use of these solutions in real projects.

3.1.2. Experimental investigations

The flexural behaviour of slim-floor beams exposed to fire has been studied through experimental testing over the last decades, although not many fire test results are available to date. The "Slimflor Compendium" published by The Steel Construction Institute in 2008 [70] collects the results of the main fire testing campaigns on fabricated slim-floor beams (SFB), asymmetric slim-floor beams (IFB) and RHS edge beams carried out between 1985 and 1996.

The first reported series [70] are SFB beams fabricated from a normalized I section with a wide plate welded to the bottom flange to produce an asymmetric section. Nine SFB specimens with precast units were tested as simple supported members with a span of 4500 mm and a heated length of 4000 mm. The fire resistance times ranged between 44 and 109 minutes, with load ratios of 0.56 and 0.42, respectively. Another six tests were carried out on SFB specimens with composite slabs constructed with deep steel decking and the same length. The fire resistance times ranged in this case between 52 and 94 minutes, with load ratios of 0.55 and 0.45, respectively. These tests were carried out at the Warrington Fire Research Center, except for three of them that were part of a two span slab specimen tested at TNO (Netherlands), spanning in this case 4.6 m.

Two standard fire tests were conducted by the Warrington Fire Research Center [65] with a SLIMDECK system using an IFB configuration. The test specimens consisted of a 280x280/180x104 rolled asymmetric beam sections. The floor was formed using 210 mm deep metal decking on top of which was cast a nominally 1 metre wide x 80 mm thick C30 grade concrete slab with a reinforcing mesh. Significant fire resistance times of 75 and 107 minutes were achieved, for load ratios of 0.43 and 0.36, respectively. Fire tests were also reported by Ma and Mäkeläinen [71] using an IFB configuration under different load ratios. It was observed that fire resistance periods over 60 minutes could be reached for load ratios under 0.5 without additional fire protection.

Three fire resistance tests were also conducted on RHS edge beam specimens [70], all of them supporting composite slabs constructed using deep composite decking. The results of these tests were reported by TNO and Corus® in the framework of a European research project funded by ECSC. Tests R1 and R3 were loaded fire tests. Test R1 consisted of a composite slab constructed using deep decking supported on two RHS edge beams. Test R3 had the same arrangement but with service openings cut into the side walls. The six specimens tested simultaneously in R2 (three test specimens each consisting of a pair of RHS edge beams) were unloaded, with the aim of obtaining only thermal data.

Recent elevated temperature tests have been carried out by the authors at the testing facilities of the Polytechnic University of Valencia (UPV) (Spain) [72], as part of a wider experimental campaign currently underway. In this experimental program, an electrical radiative furnace was used, placing the slim-floor beam specimens to be tested on top of the heating panels covering the furnace, so that they were exposed to elevated temperatures only from their lower surface. The test setup can be seen in Fig. 16. The main objective of this experimental campaign was to obtain a better understanding of the thermal behaviour of slim-floor beams and investigate the influence of different parameters over their fire performance. The fire performance of equivalent IFB and SFB sections was compared. For the SFB configuration, a HEB200 beam welded to a steel plate of dimensions 360x15 mm was used, while for the IFB configuration ½ IPE450 was welded to a steel plate of 360x30 mm. In this way, the thickness of the bottom steel plate of the IFB profile was equal to the sum of the bottom plate plus lower flange thickness of the SFB. These tests provided evidences about the different thermal behaviour between SFB and IFB due to the thermal contact resistance at the gap between bottom plate and lower flange of the steel profile.

A series of design recommendations were developed from the results of this experimental programme. It was found that an effective way to improve the fire bending capacity of slim-floor composite beams consists of acting at the bottom steel plate through the use of materials that delay the temperature rise. The comparison between IFB and SFB slim-floor configurations revealed that the SFB typology shows a better fire performance due to the thermal gap which appears between these two steel plates, providing a delay in the temperature increase of the lower cross-section parts.

3.2. Innovative composite beams

Although in the case of steel-concrete composite beams, as compared to composite columns, there is not so much opportunity of "moving" the main resisting steel parts of the beams towards the protected areas of the section (i.e. the upper parts of the beam, which are less exposed to the heat source by being protected with the surrounding concrete) and thus the lower flange would always be receiving the fire action directly, some strategies may be found for trying to increase the fire endurance of these exposed parts.

A possible action in this direction consists of increasing the previously mentioned thermal gap which appears between the two steel plates in SFB configuration. This effect has been recently proved by testing a SFB specimen in which the gap between the steel profile lower flange and the bottom plate was "physically" increased by placing a wire between these two cross-section parts (Fig. 17). In this figure, the thermocouple measurements at certain sectional locations of two SFB specimens with the same dimensions (HEB200 with bottom steel plate of 15 mm), one with lower flange and steel plate in direct contact (A1) and the other with a physical gap created between both parts by placing a wire of ϕ5 mm (A6) are compared. As it can be seen, the specimen with the intermediate wire experiences a higher difference of the temperature measurements between the bottom plate (TC1) and lower flange of the steel profile (TC4). This means that the thermal gap in this case is higher, which in turn would lead to a slower temperature rise along the steel profile and thus to a higher bending capacity in fire. Therefore, the effect of the thermal gap is favourable in terms of delaying the temperature rise at the composite section and lengthening its fire endurance.

Previous numerical investigations carried out by the authors [73] confirmed that the fire resistance of composite beams embedded in floors can be significantly enhanced by splitting the lower steel flange of an IFB section into two steel plates, generating the so-called SFB type. The thermal gap between the lower flange and the bottom plate delays the temperature rise and therefore lengthens the fire response of the beam for the same load level, as compared to the IFB configuration.

Fig. 18 shows the temperature field of two equivalent SFB and IFB configurations exposed to standard ISO834 [74] fire curve for 120 min. It can be observed that, while the IFB bottom plate reaches 940 ºC after 120 minutes of fire exposure, the lower flange of the SFB steel profile remains at a lower temperature of 825 ºC, what proves the significant influence of the thermal gap.

Fig. 19a) shows the temperature evolution along the fire exposure time of the IFB bottom steel plate and the SFB lower flange. As can be seen, due to the thermal gap in SFB, the temperature difference is maintained around 100-120ºC, showing however a moderate decrease at high fire exposure times caused by thermal inertia. It can be observed in Fig. 19b) that for the higher load levels the increase in terms of flexural capacity (relative load level referred to the room temperature capacity) obtained with the SFB configuration is quite significant (with up to a 50% increase at 0.8 load level), and this difference decreases progressively with the reduction of load level until the capacities of both configurations are very similar. It can be therefore concluded that the thermal gap becomes less influential as the fire exposure time increases.

Specifically for the SFB configuration, it was observed in previous numerical investigations [73] that the bottom plate thickness has a certain influence over the fire response of the composite beam. For the same steel profile dimensions, three different thicknesses of the bottom plate were studied, the results being displayed in Fig. 20: SFB0 (15 mm), SFB1 (10 mm) and SFB2 (20 mm). It can be observed that specimen SFB2 with the thickest bottom plate shows a superior fire behaviour, with a curve showing higher fire resistance times for any load level. On the contrary, SFB1, with the thinnest bottom plate shows the worst fire behaviour. For instance, for a 0.4 load level applied, specimen SFB1 attains 65 minutes of fire resistance, while SFB2 reaches a fire resistance of 80 minutes. The improved fire performance shown by the specimen with the thickest bottom plate (SFB2) can be explained through its lower section factor, meaning that this specimen exposes a lower surface to the fire per unit area.

3.3. Advanced materials

The beneficial properties at elevated temperatures of the advanced materials exposed in Section 2.3 can be also used for enhancing the fire performance of slim-floor beams. In particular, the applicability of high strength steel (HSS), stainless steel (SS) and lightweight concrete (LC) to slim-floor beams is been studied in this section.

3.3.1. High strength steel

Previous research by the authors [75] has shown that the use of high strength steel is favourable under fire loading, provided that it is placed at the steel profile rather than at the bottom plate. The increase of the steel grade of the bottom plate does not provide a significant increase of the bending capacity, as it results directly exposed to fire and thus its strength is rapidly affected by high temperatures. However, using HSS at the steel profile can be useful for increasing the fire bending capacity of slim-floor beams.

As can be seen in the example of Fig. 21, using HSS of grade S960 increases the bending moment resistance for fire exposure times lower than 30 minutes, regardless the position of HSS - bottom plate or inner profile -. Nevertheless, placing HSS at the bottom plate does not result effective for fire exposure times higher than 60 minutes. In turn, placing HSS at the inner profile maintains the bending moment improvement for higher fire exposure times.

3.3.2. Stainless steel

As commented previously, the performance of stainless steel under fire conditions has been assessed through extensive research over the last years [45], showing a better strength retention at elevated temperatures and a lower emissivity, which may delay the cross-section heating. Being aware of this potential, experimental [76] and numerical [77] investigations have been carried out for testing the fire performance of slim-floor beams using stainless steel, proving that the use of this material offers a considerable increase in fire resistance as compared to traditional composite beams made of carbon steel. In particular, it results convenient to locate the stainless steel part at the bottom plate of slim-floor beams, in order to take advantage of its enhanced mechanical properties in fire. This can be clearly seen in Fig. 22, where the evolution of the bending moment capacity at elevated temperature of a reference SFB specimen consisting of a HEB200 with bottom steel plate of 15 mm (A1) is compared with another specimen with the same dimensions and bottom plate made of stainless steel. A significant enhancement in terms of bending capacity is observed for all the standard fire periods (i.e. 64% relative increase at 60 minutes, 74% relative increase at 120 minutes).

Apart from a better fire performance, the use of stainless steel in slim-floor beams may also provide an improved durability and aesthetic finishing. Despite its higher initial cost, this may be overcome by a more rational use of the materials at the composite section.

3.3.3. Lightweight concrete

The use of lightweight concrete in the slim-floor encasement has been also assessed in previous investigations [73], [75] concluding that, for this typology of composite beam, the advantage provided by this type of concrete depends on the degree of reinforcement.

In unreinforced slim-floor beams, lightweight concrete does not provide any improvement in terms of the fire behaviour. This low influence of the concrete type can be explained due to the fact that the bending behaviour of the composite beam at elevated temperatures is primarily governed by the fire performance of the steel parts placed at the bottom of the cross-section, which is in tension. The lower thermal conductivity of lightweight concrete and its consequent delay of the temperature rise in the concrete mass causes a localized temperature increase in the bottom steel plate and thus a reduction of its contribution to the bending moment capacity.

However, in the case of reinforced slim-floor beams, lightweight concrete provides an additional heat insulation for the reinforcing bars and therefore increases their mechanical contribution in fire. Thus, in those cases where the amount of reinforcement in the composite beam is significant, this additional protection offered by lightweight concrete may counteract the unfavourable effect of the reduction of strength of the bottom plate and helps increasing the total bending capacity of the cross-section in fire. As it can be seen in Fig. 23, the fire resistance of a SFB specimen with normal weight concrete (SFB0) is lengthened slightly by adding reinforcing bars (SFB5), while using the reinforcing bars in combination with lightweight concrete (SFB6) provides and enhancement of more than 30 minutes at 0.3 load level.

4. CODE PROVISIONS AND DEVELOPMENT OF NEW CALCULATION METHODS FOR FIRE DESIGN

This section reviews the currently available calculation methods for the fire design of the two types of composite elements studied in this paper: concrete-filled steel tubular columns and slim-floor beams, where the authors have focused their research during the last years, being involved in specific technical committees from CEN/TC250/SC4 for the development of new calculation rules in fire.

4.1. Concrete-filled steel tubular columns

There are a number of design codes and specifications worldwide that address the design of concrete-filled steel tubular members subjected to fire.

The Chinese Code DBJ13-51 [78] establishes an equation to calculate the thickness of the external fire protection required to achieve a certain fire resistance time and is based on a research carried out by Han et al. [15].

Another approach, which is in use in North America, was developed by Kodur and co-workers [5] and has been incorporated into the National Building Code of Canada [79], ASCE/SFPE 29-99 [80] and ACI 216 [81]. This approach consists of a single design equation, which includes the main parameters affecting the fire resistance of CFT columns.

In Europe, the most extended methods for calculating the fire resistance of CFST columns are those included in EN 1994-1-2 [82], comprising three levels of design: a) tabulated data, b) simple calculation models and c) advanced calculation models. Option a) is available in Clause 4.2.3.4 in the form of a selection table which provides the minimum cross-sectional dimensions and reinforcement ratio that a CFST column must have in order to achieve a rated standard fire resistance time under a certain load level. This approach is the most simplistic and its results are highly conservative.

A specific method for unprotected CFST columns is also given in Annex H of the same code. However, this method was found unsafe for slender columns [12] - which is a frequent situation for columns in car-parks, high-rise, commercial or industrial buildings -, leading to the approval of an amendment by the European Committee CEN/TC250/SC4 which limits the relative slenderness of the column at ambient temperature to 0.5 in the application of Annex H.

The research group of the Polytechnic University of Valencia (UPV) (Spain) led by Prof Romero has been involved during the last years in the development of a new proposal for a simple calculation method of CFST columns in fire, coordinating a specific Project Team SC4.T4 appointed by the European Committee of Standardization (CEN) for the redefinition of the current Annex H.

In the previously mentioned European project FRISCC funded by RFCS [16], coordinated by the UPV research group, an extensive experimental and numerical database was generated for stablishing the basis for the development of this new simplified design method [83]. The new method solves the shortcomings of the current Annex H of EN1994-1-2 [82]. It also includes innovative shapes such as elliptical hollow sections and provides safe predictions for columns with relative slenderness at ambient temperature up to 2. It is also valid for large eccentricities, extending the current scope of Annex H. Fig. 24 shows a comparison between the predictions of the new Annex H method and the current EN 1994-1-2 method against the results of real fire tests, where it can be verified that the new method is more accurate and safe.

A key contribution of the new simplified calculation method, based on the initial proposal by Espinós et al. [84], is to assume that the effects of non-uniform temperature in the CFST cross-section can be represented by an equivalent uniform temperature for each of the different components (steel tube, concrete infill, reinforcement) of the CFST cross-section (see Fig. 25). Simplified equations were developed for providing practitioners with equivalent temperatures that can be used for evaluating the capacity of the columns in fire without the need of performing advanced heat transfer calculations. Additionally, specific flexural stiffness reduction coefficients for the steel tube, concrete core and reinforcing bars were proposed for evaluating the effective flexural stiffness of CFST columns at elevated temperatures.

The new simplified calculation method for CFST columns has been recently revised and extended by the Project Team CEN/TC250/SC4.T4 to redefine the current Annex H and, after the recent approval of the final draft, it will be available in the next generation of the Eurocodes. Additionally, the simple calculation method has been further extended for its application to different bending moment distributions [85], ranging from single curvature to double curve bending. The proposed new design method is applicable to concentric load and uniaxial bending and is in line with the cold design method in EN1994-1-1 [86], making use of interaction diagrams for elevated temperature, as in the example given in Fig. 26.

In Australia and New Zealand, a similar approach to that proposed by the UPV group for the new EN 1994-1-2 Annex H method has been adopted in the recently approved standard AS/NZS 2327:2017 [87]. In particular, the expressions proposed by Espinós et al. [84] for the equivalent temperatures of the different components of the CFST cross-section and the flexural stiffness reduction coefficients were adopted, as well as the specific buckling curves depending of the percentage of reinforcement proposed by the same authors for the case of reinforced columns in [88]. However, the effective buckling length factors in the fire situation are different in AS/NZS 2327 and EN 1994-1-2, being 0.7 and 0.85 in AS/NZS 2327, for fixed-fixed and fixed-pinned columns respectively, in contrast with the values of 0.5 and 0.7 used in the European code. Note that, similarly to the Australian approach, in the UK National Annex to Eurocode 4 (BS NA EN 1994-1-2) the effective buckling length factors were also conservatively increased to 0.7 and 0.85 [7], therefore this is an aspect that should be harmonised between the different standards. An effective length factor value of 0.9 was proposed by Ukanwa et al. [89] to be more representative for continuous CFST columns. This effective length factor was suggested from examination of the lateral deformed shape of specimens tested at elevated temperature, although further research is needed.

4.2. Slim-floor beams

While simplified models are available for partially encased and non-encased composite beams in EN 1994-1-2 [82] Annex F and Clause 4.3.4.2.2, respectively, this standard does not provide any simplified model to evaluate the fire behaviour of slim-floor beams. Project Team SC4.T5 has been appointed by the European Committee for Standardization (CEN) for the development of design rules for shallow floor beams in fire and their integration into the next generation of the Eurocodes, work which is currently underway.

In the absence of any specific method for assessing the temperature development in slim-floor beams exposed to fire, different proposals have been developed during the last years in order to provide models that allow predicting the temperature field in slim-floor composite beams.

Zaharia and Franssen [90] developed simple equations for the calculation of temperatures within the cross-section of an IFB, providing formulas for the assessment of the temperature at the bottom plate, web of the steel profile and reinforcing bars. Cajot et al. [91] defined a set of formulas to determine the thermal field in slim-floor beams based on the existing equations in EN 1994-1-2, with particular assumptions for IFB and SFB configurations. Romero et al. [92] compared the previous simplified models and defined a methodology for the evaluation of temperatures based on the existing formulas in EN 1994-1-2 for the different parts of the slim-floor cross-section combined with the use of the Zaharia and Franssen equations for the prediction of the lower flange temperature. More recently, Hanus et al. [93] proposed specific analytical equations to predict the temperature of longitudinal reinforcing bars embedded in slim-floors. From the literature review, it was noticed that the prediction of the temperatures of the longitudinal reinforcing bars are the point where the developed thermal models mostly focus on. In fact, different models exist for the reinforcing bars temperature assessment, while for the bottom plate and web profile prediction the model from Zaharia and Franssen [90] is widely accepted.

Based on the previous investigations, a simplified approach for the evaluation of the plastic bending moment of a slim-floor beam after a certain period of fire exposure can be given, consisting of the subdivision of its cross-section into different zones with representing temperatures. In particular, 7 zones can be defined, as indicated in Fig. 27. Zone 1 considers the lower flange of the steel profile plus a portion of the bottom steel plate with the same width. In turn, zone 2 comprises the outermost areas of the bottom plate. In turn, the web of the steel profile is divided into two parts: zone 3, with temperatures over 400ºC and zone 4, below 400ºC. Zone 5 corresponds to the upper flange of the steel profile, while zone 6 is the top concrete compression area and zone 7 includes the longitudinal reinforcing bars.

The previously described simplified models provide equations for obtaining the temperature at each of the denoted parts. Once these temperatures are known, the corresponding strength reduction factors for the different parts can be obtained from EN 1994-1-2. Using the reduced mechanical properties at elevated temperature, the position of the plastic neutral axis (PNA) and the value of the plastic bending resistance of the cross-section can be finally computed by applying the equilibrium equations.

A more precise approach to evaluate the flexural capacity of composite beams at elevated temperature consists of discretizing the cross-section for evaluating the realistic temperature field (i.e. by means of a heat transfer sectional model) and afterwards applying a fibre-based model for computing the ultimate bending moment by equilibrium. In a first instance, the cross-section is meshed and a heat transfer analysis is conducted, an example is given in Fig. 28.

Each cell of the mesh is then characterized by its position and its temperature. Using the reduced mechanical properties of steel and concrete at the representative temperature of each cell, the contribution of each fibre to the axial force is computed and the position of the PNA is determined by equilibrium. Once the PNA location for a given temperature is known, the plastic bending resistance of the cross-section can be easily computed by taking moments from each fibre. This approach is similar to the previously described simplified model, but in this case the finer discretization of the cross-section allows for a more accurate assessment of the bending capacity of the composite beam, providing more realistic predictions.

Additionally, the most sophisticated way to evaluate the thermo-mechanical behaviour of slim-floor beams is by means of advanced finite element (FE) models. A three-dimensional FE model developed by the authors [73] and validated against experimental results was used to assess the accuracy of the simplified temperature formulae developed by Zaharia and Franssen [90] and Hanus et al. [93]. Specifically, the temperature evolution at the bottom plate, reinforcing bars and web profile was compared [94].

The results of this comparison, for a SFB cross-section composed of an HEB200 profile welded to a bottom plate of 15 mm thickness are shown in Fig. 29. As it can be seen, the bottom plate simplified formula developed by Zaharia and Franssen provides accurate predictions of the bottom plate temperature (points T1 and T2). However, at the lower flange of the steel profile (point T3) a significant lower temperature is found through the application of this formula, since the model was developed for IFB specimens. The reason of this difference comes from the thermal gap mentioned before that appears in the case of SFB cross-sections creating a difference between the two steel parts, while for IFB configurations a single temperature for the bottom flange would be enough and more representative. Therefore, specific formulas for the prediction of the temperatures at the lower flange and bottom plate of SFB configurations are needed, as the temperature difference caused by the thermal gap is not well captured with the available simplified formulation using a single temperature for representing the lower part of the composite beam.

Additionally, the available formulae for predicting the temperature of the reinforcing bars (Tr) was also assessed. As can be seen, both models - Zaharia and Franssen [90] and Hanus et al. [93] - provide higher temperatures than those obtained through the FE model predictions. However, the model from Hanus et al. shows closer results to the realistic temperatures for 90 and 120 minutes of fire exposure. In any case, the use of the previous formulae would result on safe predictions in terms of the thermo-mechanical performance of the composite beam in fire.

Further research is therefore needed for developing more precise formulas for the temperature of the reinforcing bars in slim-floor beams.

5. SUMMARY AND CONCLUSIONS

The most recent developments for steel-concrete composite members in fire have been reviewed in this paper. In particular, the latest research on concrete-filled steel tubular columns and slim-floor beams at elevated temperatures have been presented, summarizing the main contributions of the authors, along with the current state-of-the-art on this field.

Innovative solutions that may help improving the fire performance of conventional CFST columns have been presented, such as double-skin or double-tube configurations, as well as CFST columns with embedded steel profiles or massive steel core. The superior capacity of these innovative solutions when exposed to fire has been proved by means of both numerical studies and experimental testing.

For the case of slim-floor beams, the differences between the fire performance of IFB and SFB configurations have been highlighted, and the improved fire behaviour of the latter option due to the thermal gap that appears in the fire situation between the bottom plate and the lower flange of the steel profile has been shown through numerical and experimental results.

Different ways for enhancing the fire performance of CFST columns and slim-floor beams have been presented, which may be regarded as alternative solutions to the use of external protection via intumescent coating. This is more clearly shown in Fig. 30. In steel-concrete composite members, the strategic location of the different materials offers an "internal protection" against fire, in front of the conventional "external protection". In the case of CFST columns this is attained by splitting the outer steel tube into two profiles, where the inner profile results thermally protected by the surrounding concrete. In turn, in slim-floor beams, this protection is achieved by increasing the thermal gap between the lower flange and the bottom plate in SFB configurations and acting through the insulation of the reinforcing bars embedded in concrete. These solutions may result more cost-effective along the whole life cycle of the structure than the use of intumescent coating, which requires a periodic maintenance.

The use of advanced materials, such as high strength steel, stainless steel, high strength concrete, fibre-reinforced concrete, lightweight concrete, geopolymer concrete or recycled aggregate concrete has also been considered as a way for enhancing the fire performance of these composite members. Design recommendations have been given for a rational use of these advanced materials, in order to take advantage of their improved mechanical properties at elevated temperatures or other beneficial qualities such as the reduced carbon footprint, as it is the case of GPC or RAC.

Finally, the simple calculation methods available worldwide from the main design codes as well as from the reviewed literature have been presented. For CFST columns, a simple calculation method recently developed by the authors and co-workers from the CEN/TC250 Project Team SC4.T4 that will replace EN 1994-1-2 Annex H has been presented. In turn, for slim-floor beams, a simplified approach which combines the use of adapted equations from EN 1994-1-2 and specific temperature equations available in the literature is given.

Although the potential of these isolated steel-concrete composite members under fire conditions has been demonstrated in this paper, further studies that consider the global behaviour of composite steel-concrete solutions, including the realistic modelling of the connections and the consideration of the composite effect should be carried out, in order to provide designers with a fully integrated composite construction system.

ACKNOWLEDGEMENTS

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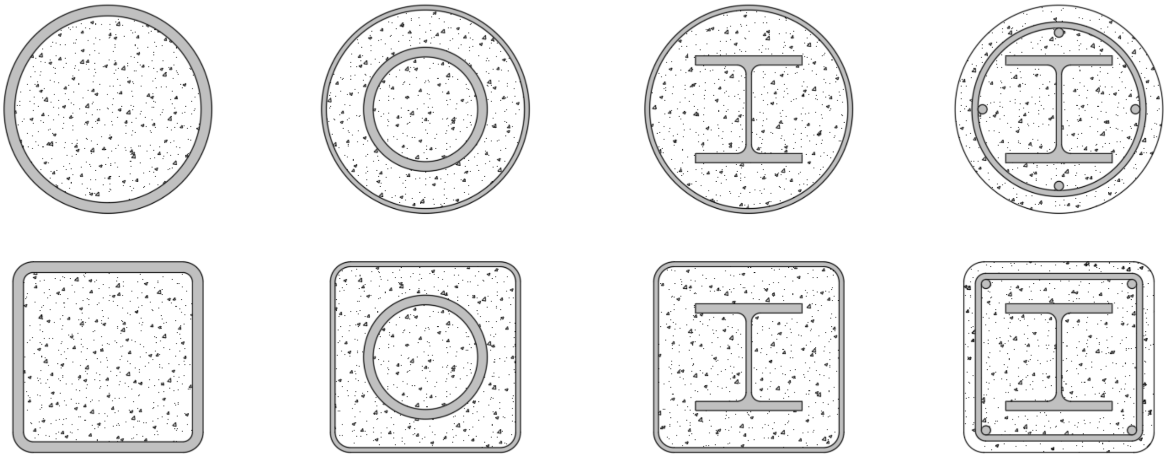


Fig. 1. Typical cross-sections of steel-concrete composite columns.

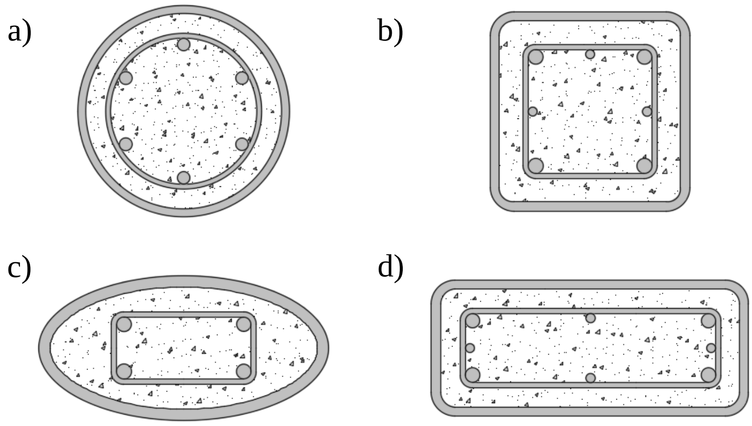


Fig. 2. Different CFST cross-section geometries: a) circular; b) square; c) elliptical; d) rectangular.

a)

b)

c)

Fig. 3. Examples of buildings using CFST columns: a) Canton Tower (Guangzhou, China); b) Peckham library (London, UK); c) Marguerite Yourcenar Media Library (Paris, France) [16].

a) b)

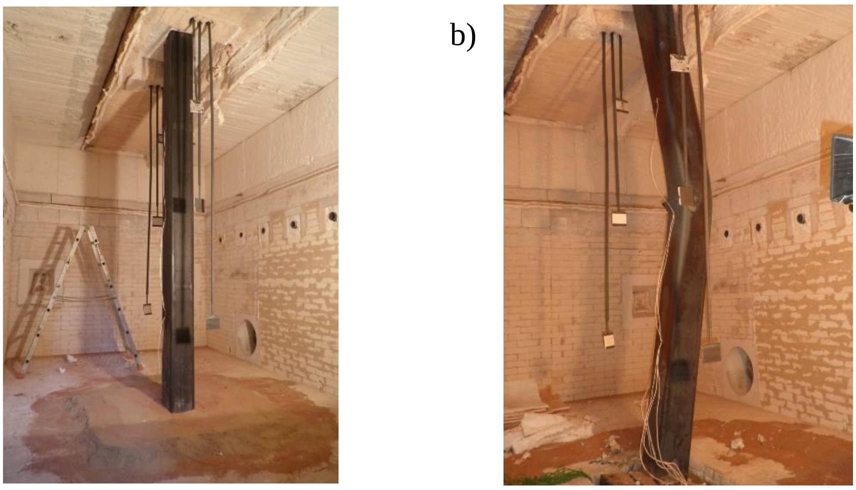
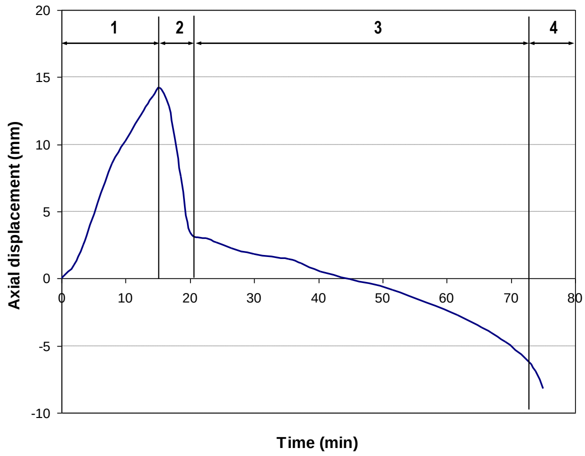
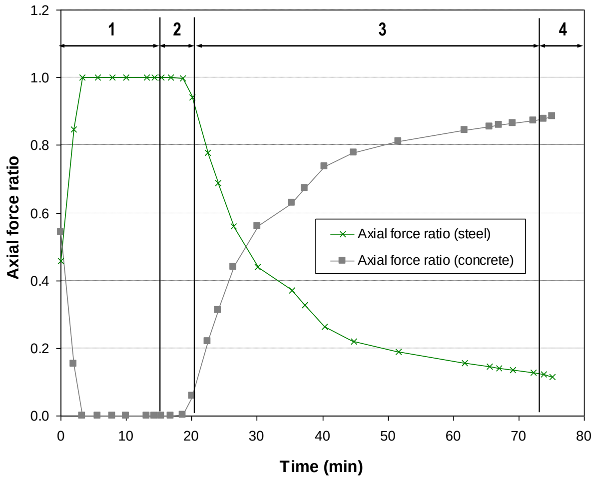


Fig. 4. View of a square CSFT column before (a) and after (b) the fire test [16].



(a) Axial displacement versus time



(b) Axial force ratio versus time

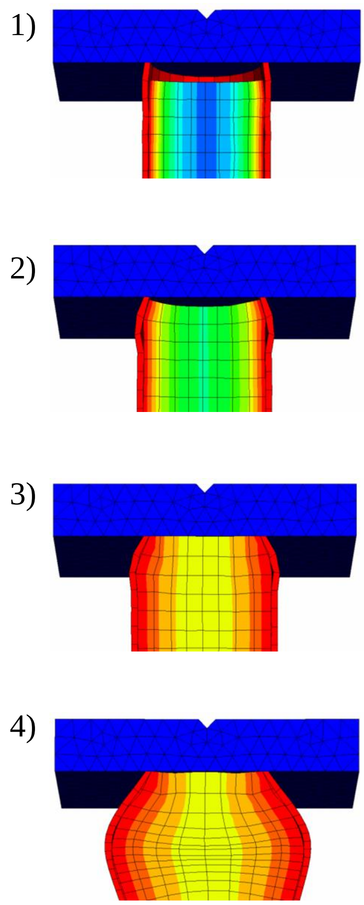


Fig. 5. Stages of the fire response of a CFST column.

a)  b) 

Fig. 6. Different types of concrete-filled dual steel tube sections: a) double-skin section; b) double-tube section [28].

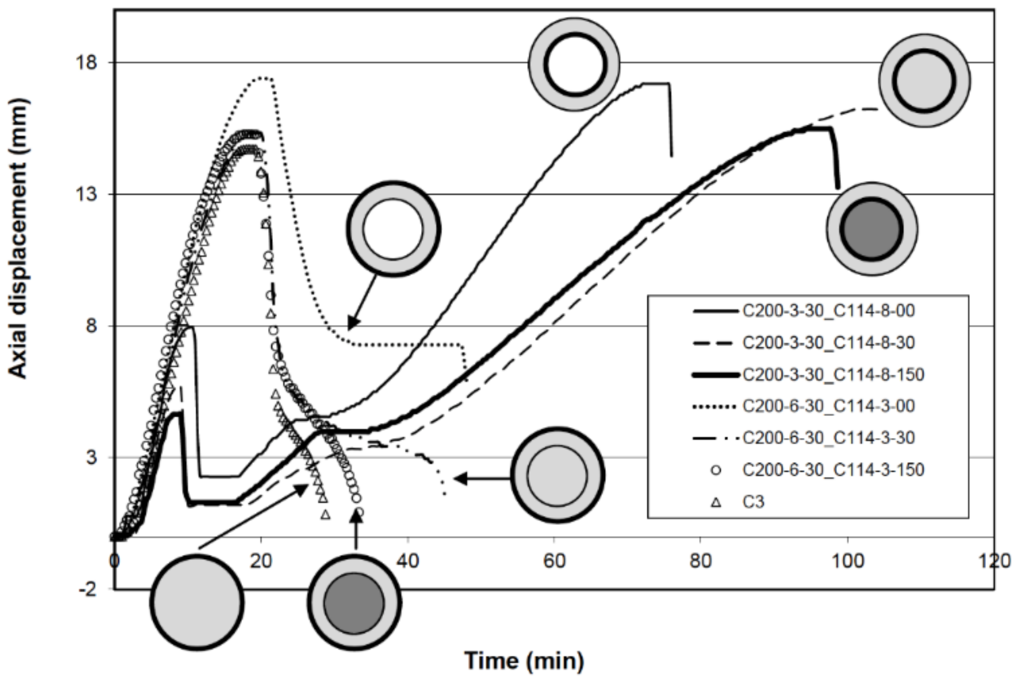
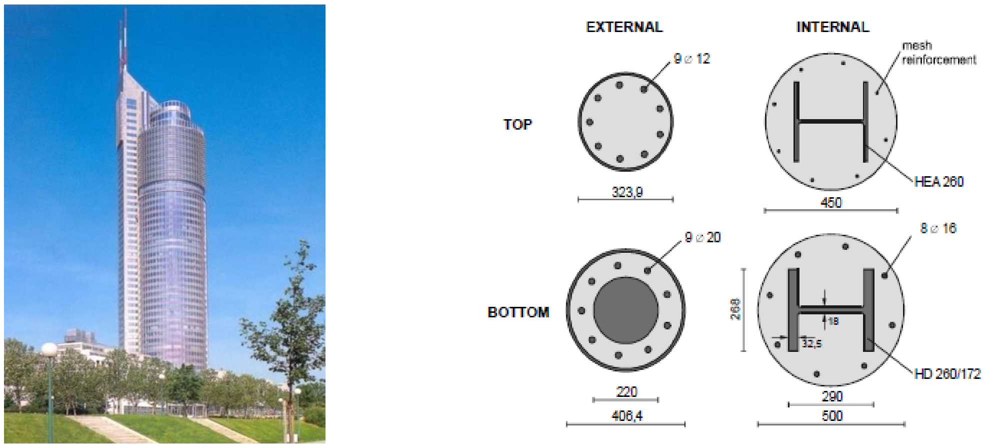


Fig. 7. Comparison of the evolution of the fire response for different concrete-filled dual steel tube columns, against a reference CFST column [28]. Legend: CDo-to-fco_CDi-ti-fci, where: Do = outer diameter, to = outer tube thickness, fco = outer concrete compressive strength (nominal), Di = inner diameter, ti = inner tube thickness, fci = inner concrete compressive strength (nominal).

a)



b)

Fig. 8. Examples of buildings using innovative composite columns: a) Millennium Tower (Wien, Austria), b) Highlight Towers (Munich, Germany) [16].

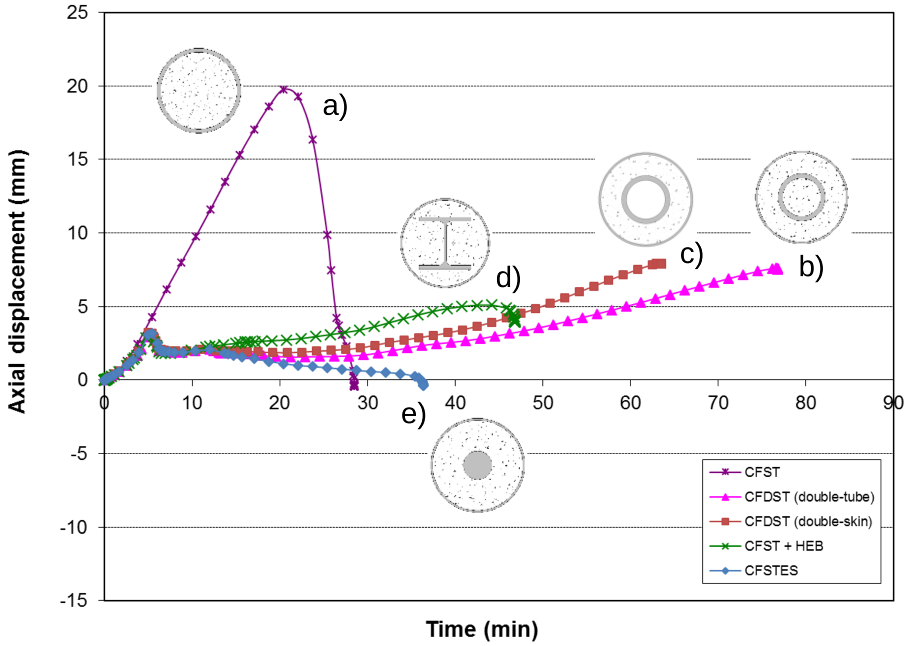


Fig. 9. Comparison of the fire response of different innovative composite cross-sections: a) CFST; b) CFDST (double-tube); c) CFDST (double-skin); d) Embedded HEB; e) Embedded steel core [32].

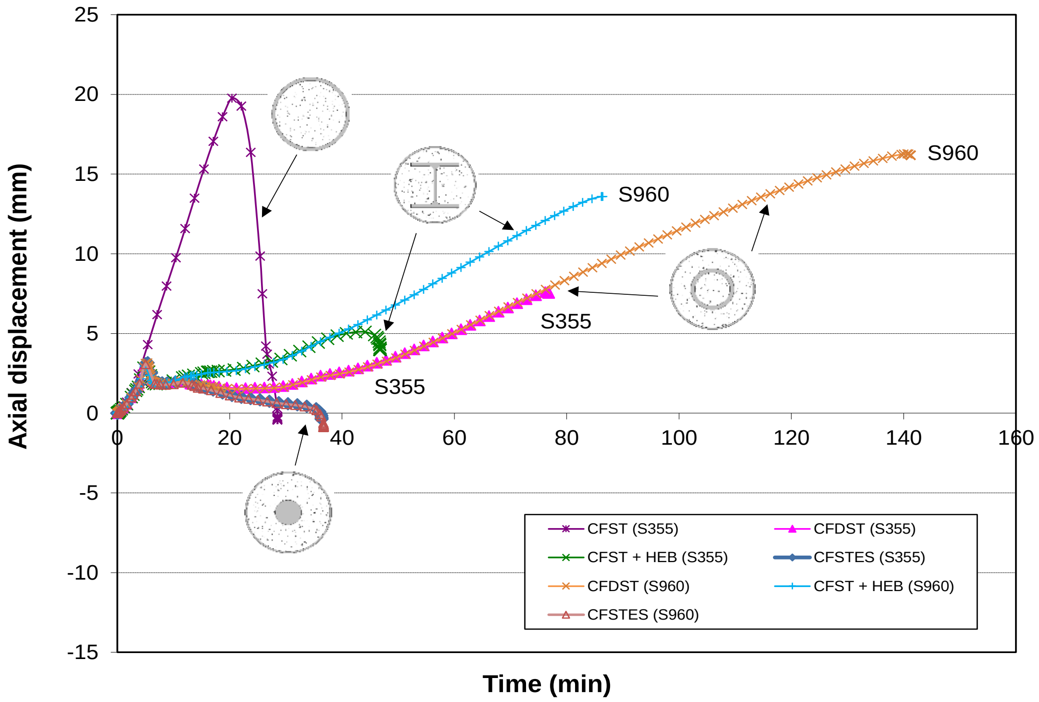


Fig. 10. Comparison of the fire behaviour of composite columns with innovative sections, using different steel grades at the inner profiles: S355 vs S960 [32].

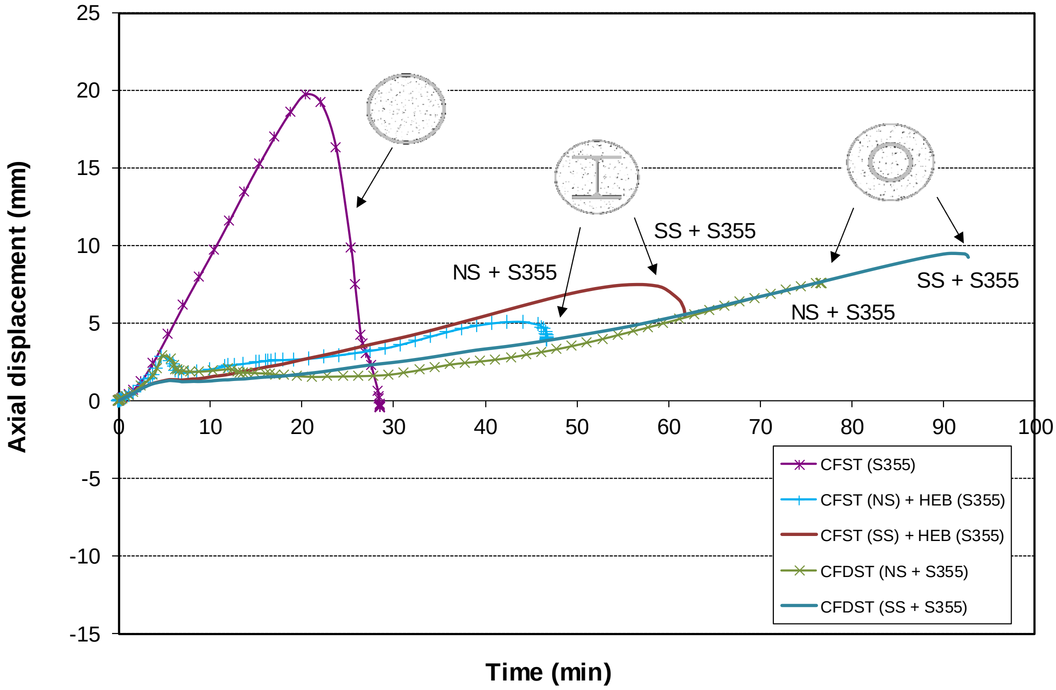


Fig. 11. Effect of using stainless steel at the outer tube (inner steel profiles of grade S355) (NS = Normal Steel, SS = Stainless Steel).

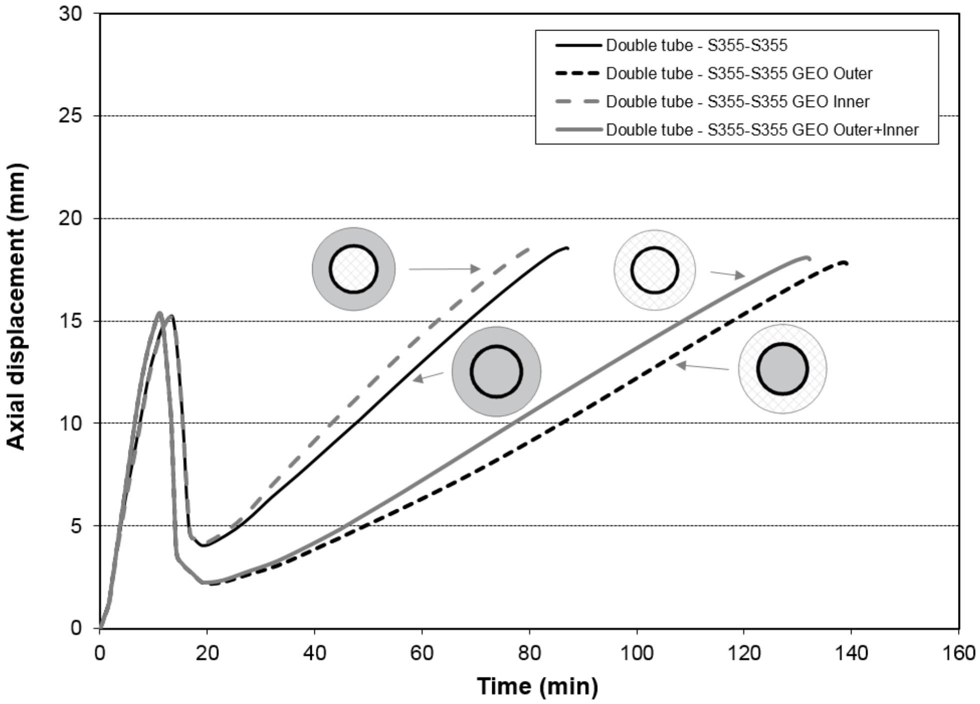


Fig. 12. Comparison of the fire response of different double-tube columns using geopolymer concrete [58] (GEO Outer = Geopolymer Concrete at the outer ring, GEO Inner = Geopolymer Concrete at the inner core).

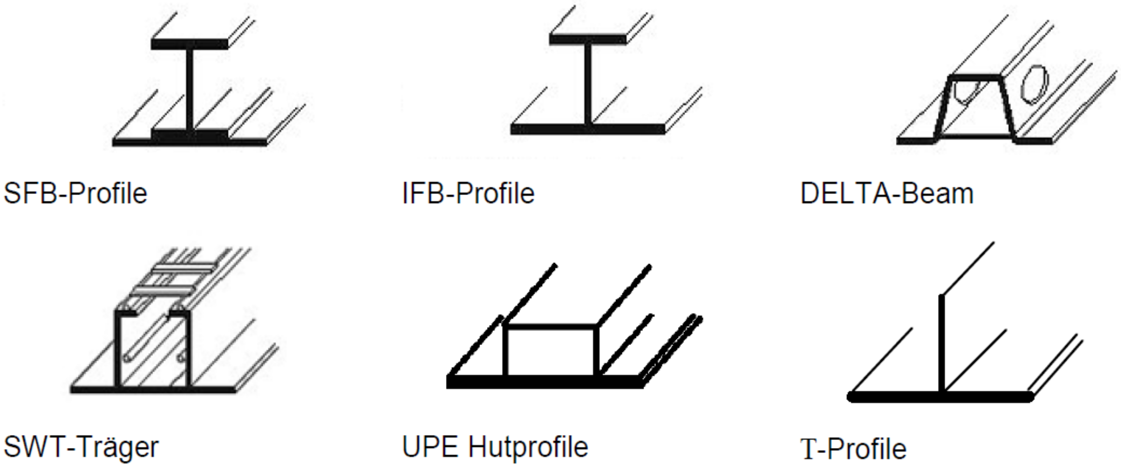


Fig. 13. Typical cross-sections of composite beams [95].

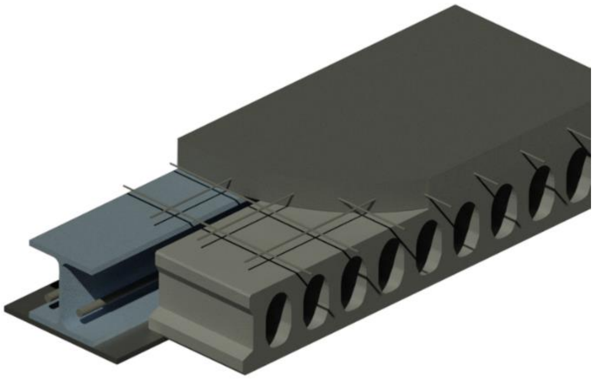


Fig. 14. Slim-floor beam with hollow core slabs. 3D general view [73].

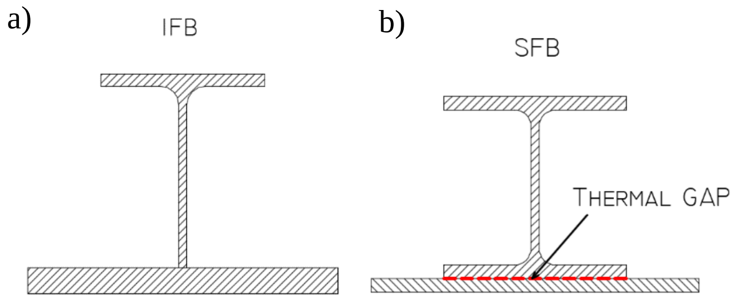


Fig. 15. Types of slim-floor beams: a) IFB, b) SFB.

Fig. 16. Experimental setup for elevated temperature slim-floor tests at the Polytechnic University of Valencia (Spain).

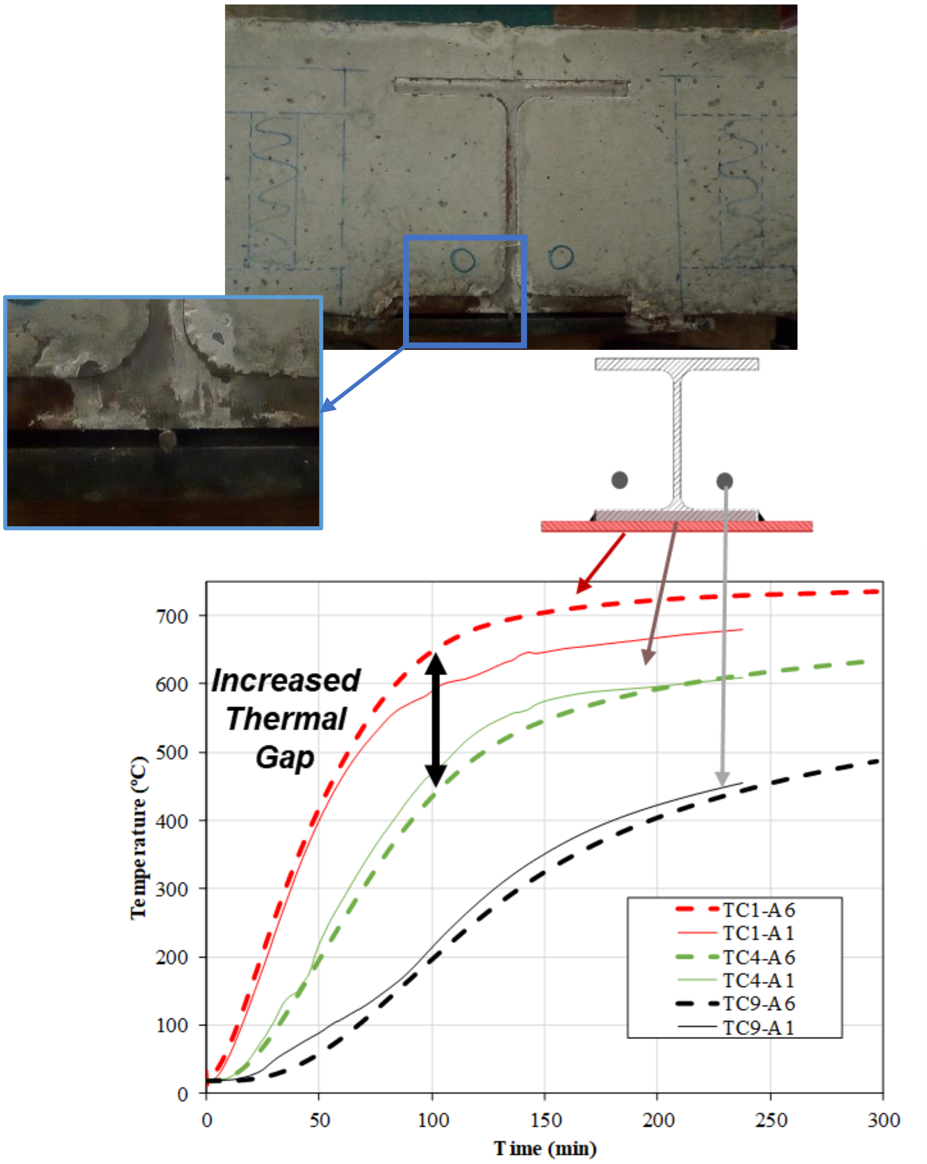


Fig. 17. Effect of increasing the thermal gap between the lower flange and bottom plate of the SFB (A1 = bottom plate and lower flange in direct contact, A6 = wire of ϕ5 mm opening the gap between the two plates).

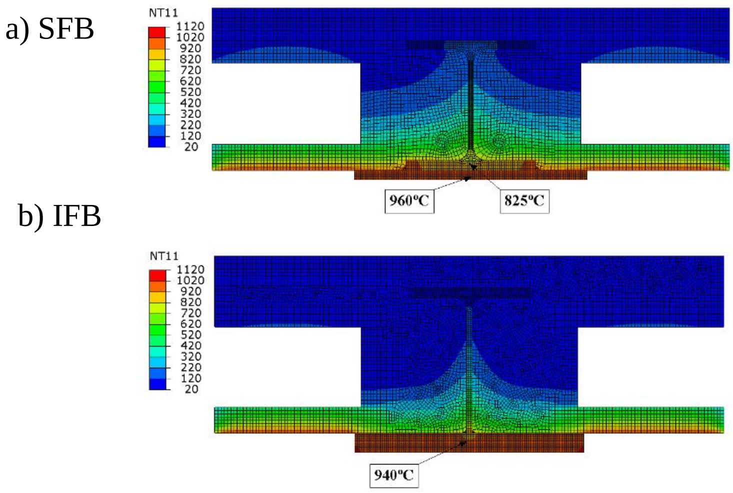


Fig. 18. Comparison of the temperature field of SFB (a) and IFB (b) configurations after 120 minutes fire exposure.

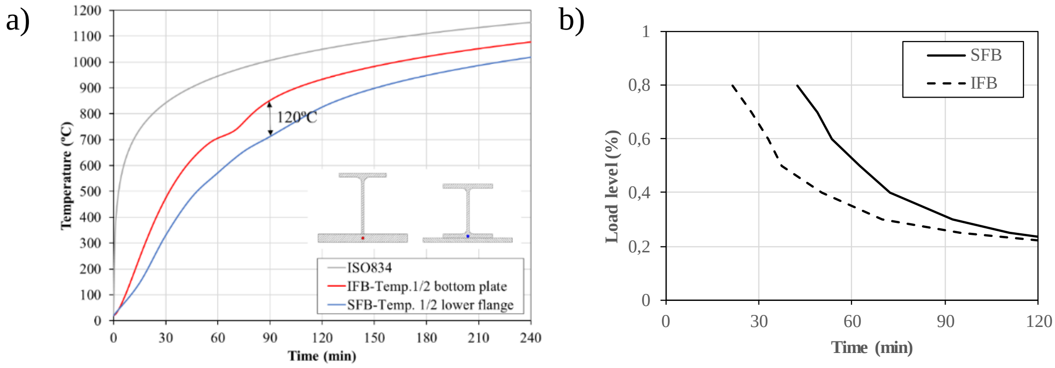


Fig. 19. Comparison of the fire performance between SFB and IFB configurations: a) Temperature evolution; b) Reduction of the flexural capacity at elevated temperature [94].

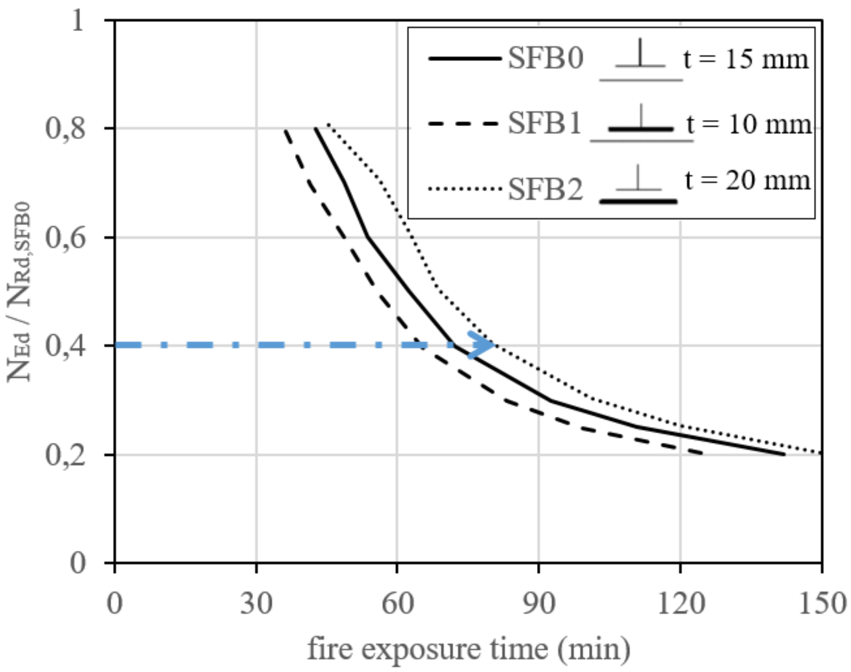


Fig. 20. Influence of the bottom plate thickness (“t”) over the mechanical response of the SFB configuration in fire [73].

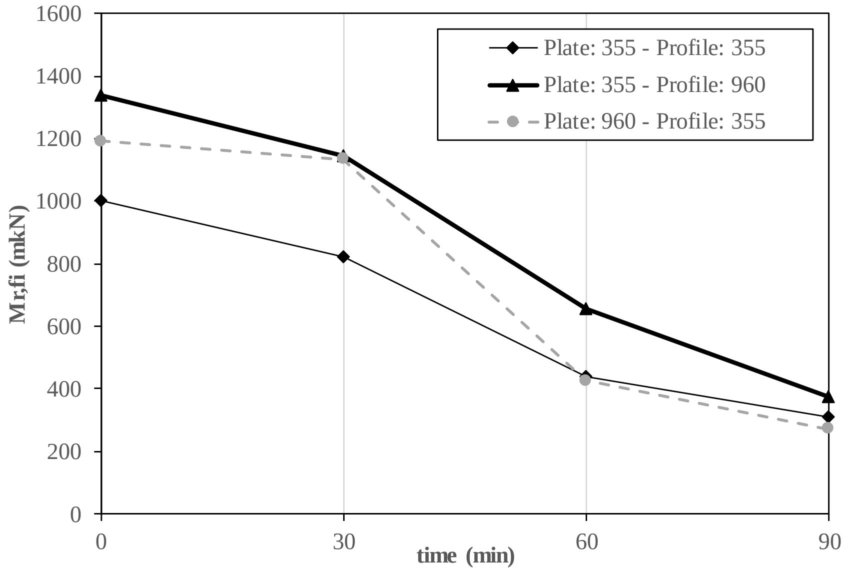


Fig. 21. Influence of the use of HSS over the evolution of the bending moment resistance of SFB specimens along the fire exposure [94] (steel grade of bottom plate and lower flange of steel profile given in the legend).

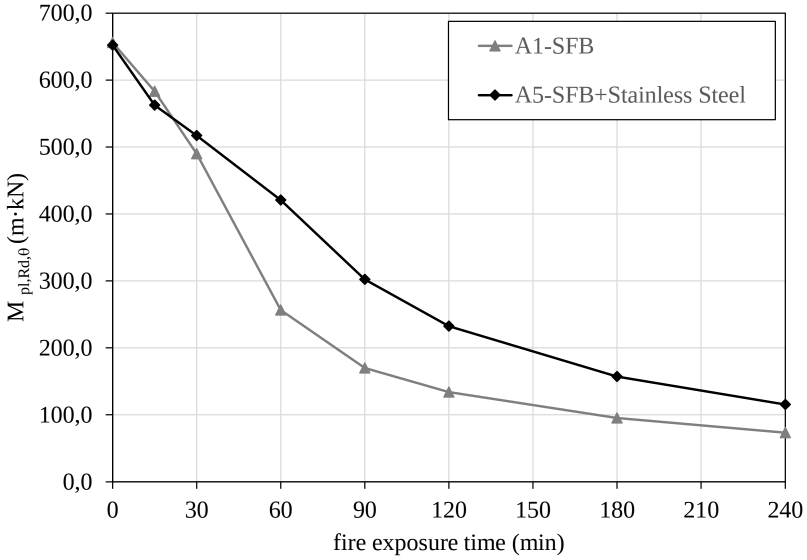


Fig. 22. Influence of the use of stainless steel at the bottom plate over the evolution of the bending moment resistance of SFB specimens along the fire exposure. (A1 = SFB with normal steel at bottom plate, A5 = SFB with stainless steel at bottom plate).

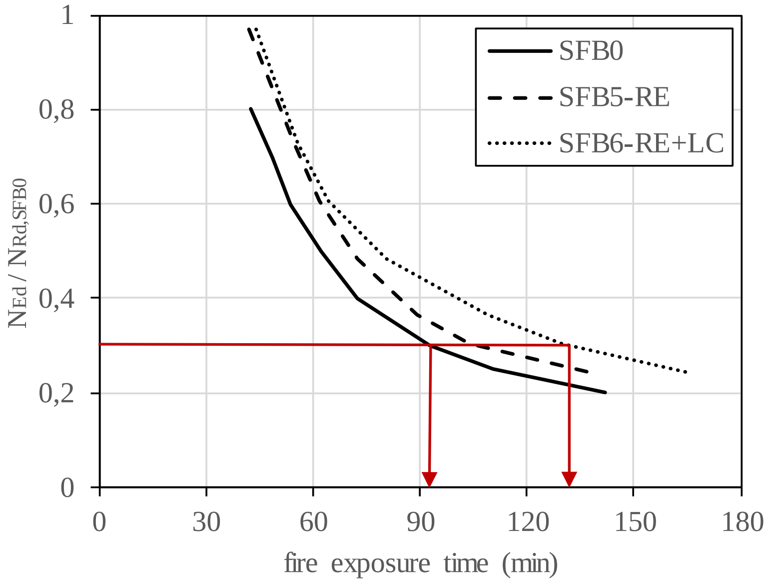


Fig. 23. Influence of the reinforcing bars (RE) and lightweight concrete (LC) over the mechanical response of SFB specimens in fire [73] (SFB0 = SFB without rebars, SFB5 = SFB with rebars, SFB6 = SFB with rebars and lightweight concrete).

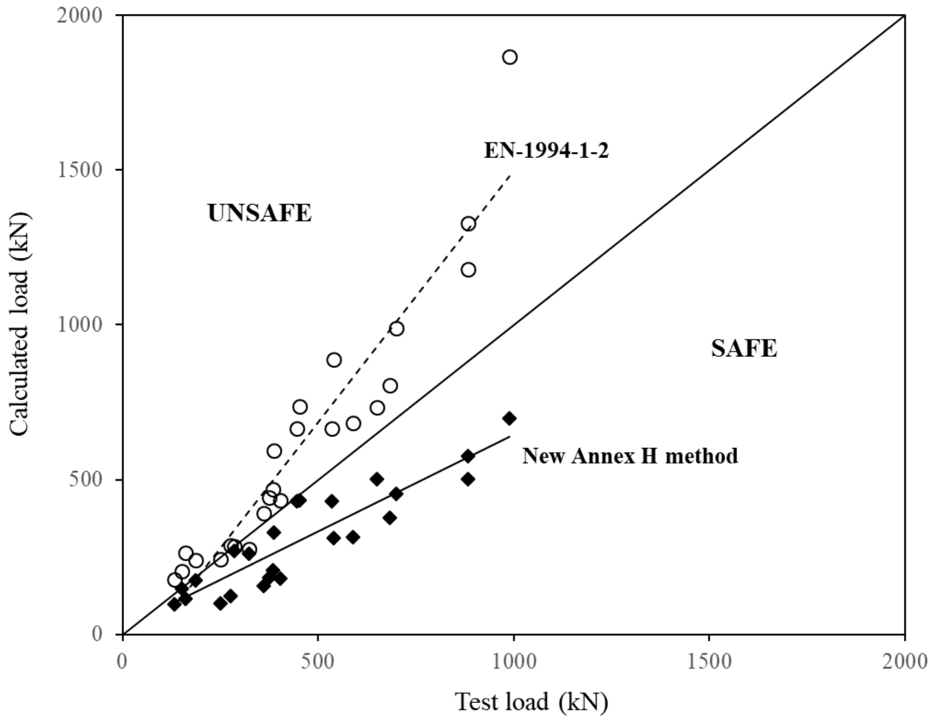


Fig. 24. Comparison of ultimate loads between current method in EN 1994-1-2, new Annex H method and tests.

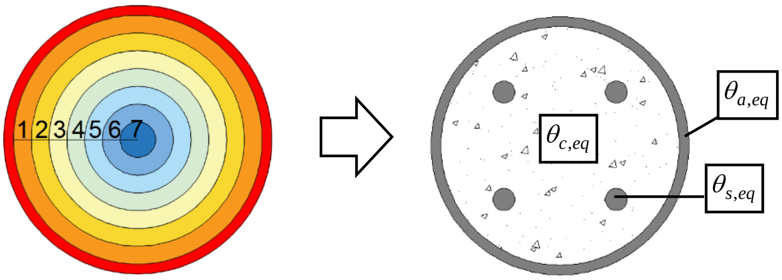


Fig. 25. Definition of equivalent temperatures within a CFST cross-section.

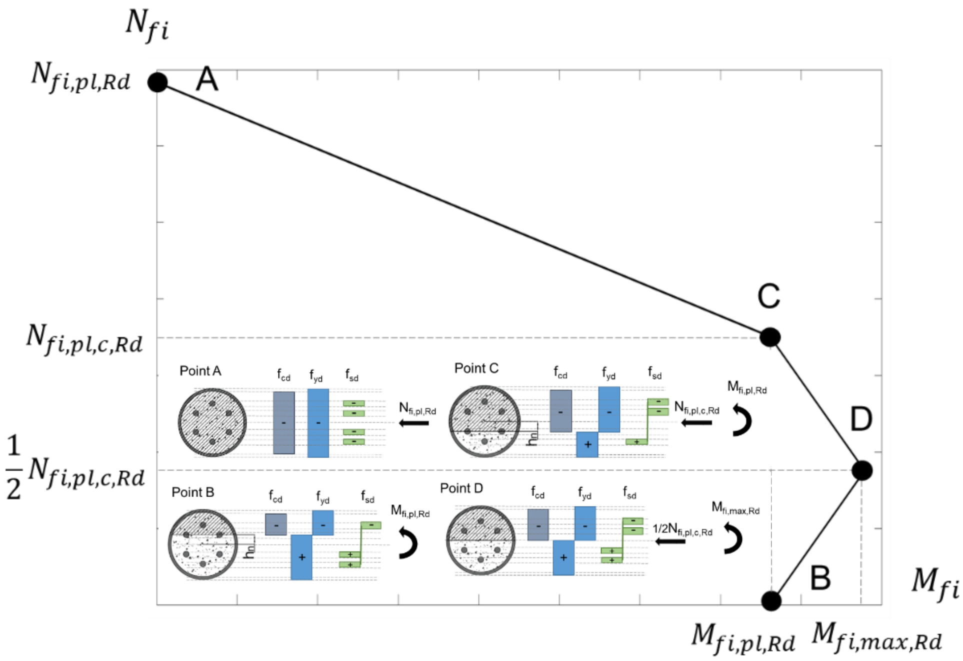


Fig. 26. Example of an interaction diagram for the calculation of eccentrically loaded CFST columns at elevated temperature [85].

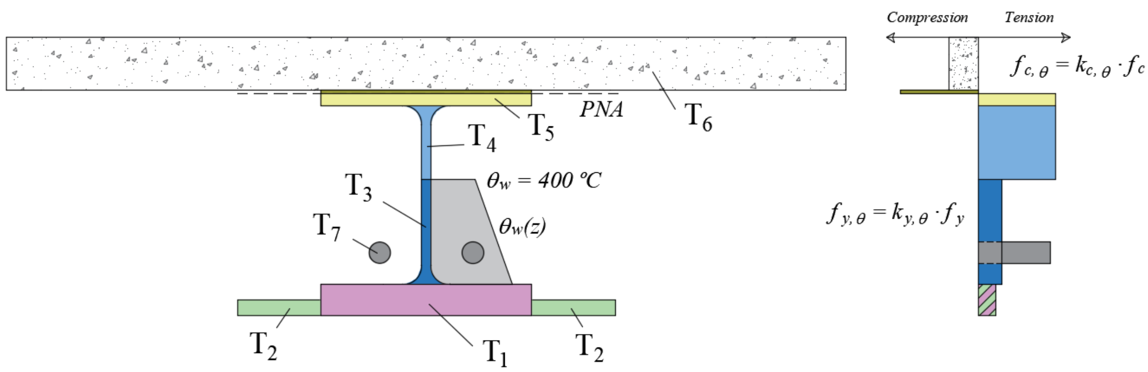


Fig. 27. Subdivision of a SFB cross-section into different temperature zones.

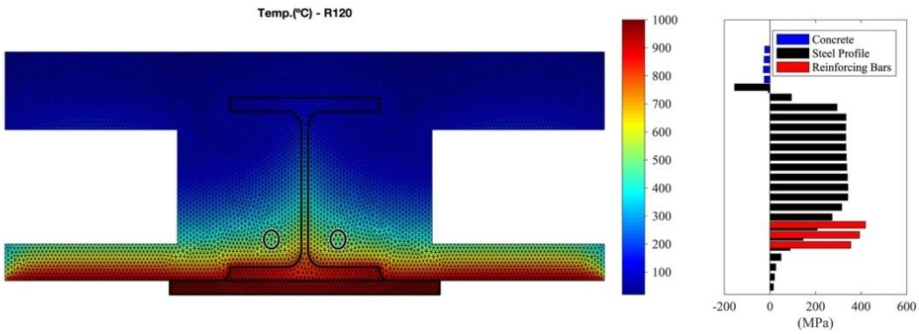


Fig. 28. Evaluation of the ultimate bending moment of a SFB cross-section after 120 minutes fire exposure by means of a fibre-based model [94].

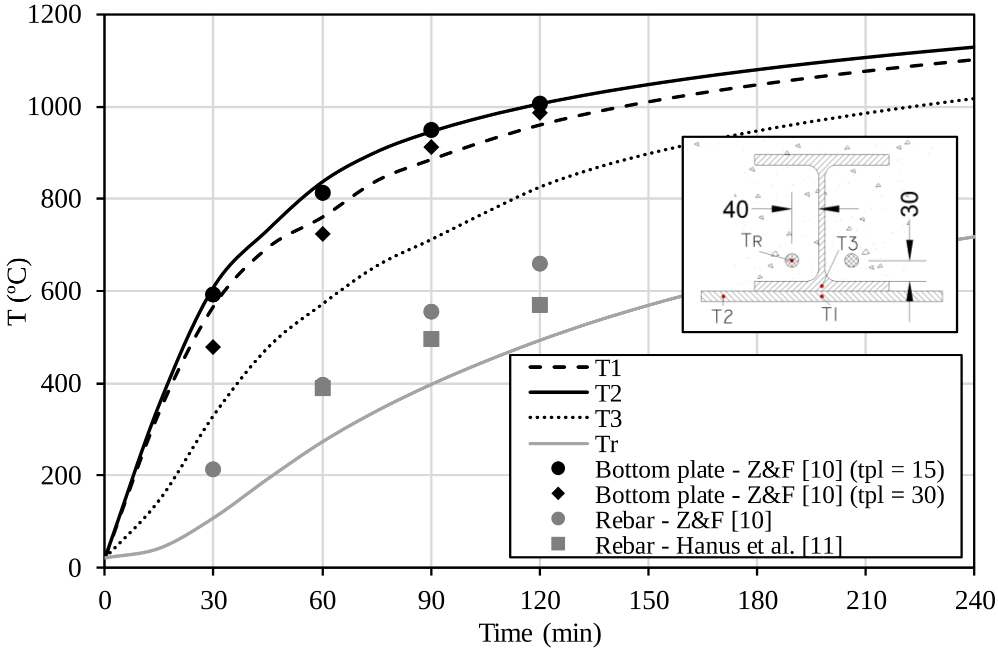


Fig. 29. Comparison of temperatures at different locations of a SFB cross-section between FEM and the available simplified thermal models [94].

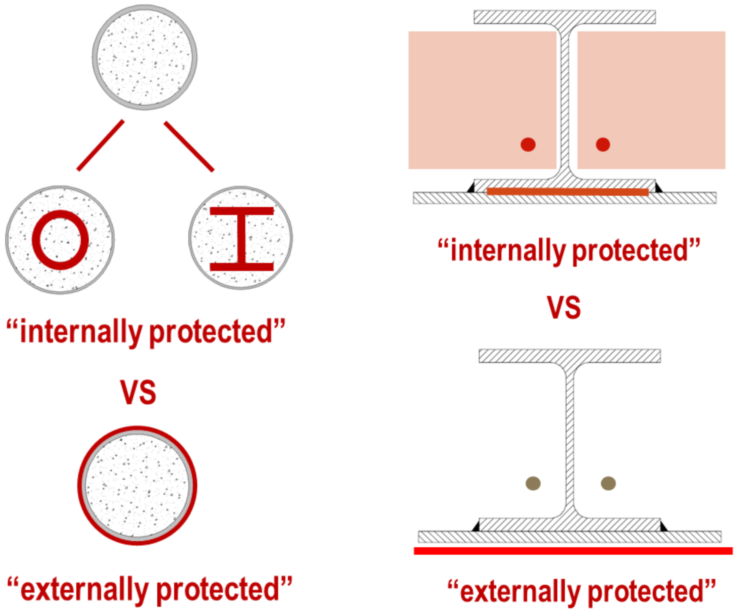


Fig. 30. Different approaches for enhancing the fire endurance of composite members (CFST columns and slim-floor beams): internal versus external protection.

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