Structural fire performance of restrained composite columns made of concrete-filled square and circular hollow sections

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Abstract

Most of the previous studies on concrete-filled steel hollow section columns at high temperatures addressed the effect of depth-to-thickness ratio, column slenderness, initial applied load level, load eccentricity, and local buckling of concrete-filled steel tubes on the fire resistance of these columns. For this reason, it important and required to study the influence of the axial and rotational restraint on the buckling behavior of these types of columns subjected to fire. The results of a series of fire resistance tests on these types of columns inserted in a steel frame are presented and discussed in this paper. The primary test parameters taken into account were column slenderness, type of section geometry, and axial and rotational restraint level imposed by a surrounding steel frame to the columns. The specimens were then uniformly exposed to the ISO 834 standard fire curve, and the critical time (fire resistance), failure temperature distribution and respective failure modes were assessed. Finally, the results of this research study showed most of all that the fire resistance of identical semi-rigid ended columns may be not significantly affected by the stiffness of the surrounding structure but, on the contrary, their post-buckling behavior may be affected.

Introduction

Since fire safety is one of the key aspects of structural design, it is essential to develop a full understanding of the fire performance of concrete-filled steel tubular (CFST) columns. Some experimental and numerical research has been carried out to investigate the fire performance of these columns in the last years. Some examples are the research works of Han *et al.* (2003),[1] Moliner *et al.* (2013),[2] and Pagoulatou *et al.* (2014).[3] The most important outcome to be stressed from the literature is that the EN 1994-1-2:2005[4] simple calculation model may lead to unsafe results for pinned-pinned columns under concentric axial loads and to highly conservative results for eccentric loads.[2,5,6] This simple calculation model may produce high errors for columns, which do not make use of reinforcing bars, but it may give accurate predictions for bar-reinforced columns.[2] Note that the concrete filling offers an attractive practical solution for providing fire protection to steel hollow columns without any external protection, since the fire resistance of this type of columns may be between 30 and 60 min when the load level is 20% of the respective design value of the buckling load at room temperature,[7] in contrast to the steel tube columns where their fire resistance is commonly less than 30 min.[8] This increase is due to the composite action between concrete core and steel tube. At first, the steel tube expands more than the concrete core, due to the higher temperature and thermal expansion of the steel, which sustains the serviceability load applied to the column. In the latter stages the steel tube starts to buckle locally, which transfers the load to the concrete core. Finally, when the concrete core loses its strength, the column buckles.[5] Moreover, comparing the fire resistance of circular and elliptical columns, it was observed that the former might have higher fire resistance than the latter. This is due to the lower section factor that the circular sections present, which delays the heating up of the column, thus providing a longer fire resistance. The difference in performance between elliptical and circular columns is less evident for slender columns, since the sectional effects have a lower influence.[9] With regard to the concrete strength, it was found out that the fire resistance of the circular CFST columns with high strength concrete was lower than the columns with normal strength concrete for the higher load levels (40% of the load bearing capacity of the columns at ambient temperature), while for the smaller load levels (20%) the columns with high strength concrete produced the same or higher fire resistance.[2] Lastly, the addition of reinforcing bars still has a favorable effect on the response of the CFST columns, although only small increments in terms of fire resistance are occasionally obtained.[6]

On the other hand, most of the studies did not take into account the interaction between the column and the surrounding building structure. The response of these columns when inserted in a building structure is different than when isolated. Restraints to the thermal elongation of the column play an important role on column's stability, since it induces different forms of interaction between the heated column and the cold adjacent structure. While the axial restraint to thermal elongation of the columns may have a detrimental effect, the rotational restraint may have a beneficial effect on the fire resistance.[10-15]

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Therefore, this research work presents and discusses the results of fire resistance tests on axially loaded CFST columns with restrained thermal elongation in order to investigate the effect of the axial and rotational stiffness of the surrounding structure to the column. Other important goals of this research work were also to evaluate the structural response of columns with different cross-sections (circular and square sections) and the slenderness of this kind of columns on their fire resistance. In each experimental fire resistance test, temperatures in the furnace and at several points of the columns, as well as vertical and horizontal displacements, rotations and restraining forces developed by thermal elongation were measured to achieve those objectives. Finally, another purpose of this experimental research was to provide valuable data for the validation of numerical models, which may help develop a suitable analytical guidance in the design of restrained CFST columns subjected to fire, which is dependent on all studied parameters in this research work.

Materials and Methods

Test program

The experimental tests on axially loaded CFST columns with restrained thermal elongation were conducted at the Laboratory of Testing Materials and Structures of the University of Coimbra, Portugal. The experimental program consisted of eight fire resistance tests, four of which were performed on columns under a 30 kN/mm axial restraint to their thermal elongation and a 94,615 kN.m/rad rotational restraint, and four others under a 110 kN/mm axial restraint and a

131,340 kN.m/rad rotational restraint. These values were similar to those previously used in this Laboratory for analogous studies[12,13] to facilitate comparisons. The lower value means practically absence of thermal restraint, whereas the higher stiffness tried to simulate a common two-story building of 3×4 bays of 6 m span. This experimental program is summarized in Table 1.

Test specimens

Two different types of composite columns were selected for this study: circular and square columns (Figure 1). All columns were made of hollow steel profiles completely filled with reinforced concrete. All reinforcing bars used in the test specimens were made of B500 structural steel and all specimens present a similar concrete composition with calcareous aggregate and C25/30 class, according to EN 1992-1.1:2004.[16] In addition, all steel profiles were 3.15 m tall and were made of S355 structural steel (with a nominal yield strength of 355 MPa and a tensile strength of 510 MPa), according to EN 1993-1.1:2004.[17] Prior to concrete casting, the longitudinal steel reinforcement bars and a steel hook were welded at a S355 steel plate, measuring 450×450×30 mm, which was then welded to the section extremity of the steel tube as well. After concrete curing, it was checked that the concrete surface was flush with the steel tube at the top in order to ensure full contact between the top concrete surface and the end steel plate (which was also welded to the top of steel tube). Note that both ends of the longitudinal steel reinforcement bars were welded to the steel end plates of the columns. Another important point to note is that three to five days after concrete casting, the moisture content of the concrete was about 4.5%, according to the procedure described in EN 1097-5:2009.[18] This parameter was measured as soon as possible because the water loss from the concrete inside the steel tubes is too limited.

For each specimen, the transversal reinforcement was performed by 8-mm diameter stirrups with a spacing of 150 mm until about 800 mm from the supports, and with a spacing of 200 mm in the central part. The concrete covering related to the stirrups for all test columns was about 25 mm. For each type of column, two different cross-sections were used. Therefore, the circular hollow steel sections were 193.7 and 273.0 mm in diameter and the square ones were 150 and 220 mm wide. The widest circular and square specimens had eight longitudinal steel reinforcing bars, four of which were 16 mm in diameter and the others were 10 mm, and the steel tube had a 10-mm wall thickness; whereas, the narrowest specimens had four longitudinal steel reinforcing bars with 12 mm in diameter and a steel tube with a 8-mm wall thickness. All columns had similar longitudinal reinforcement ratios. Also, the widest steel rebars were placed at the corners of the cross-sections and the others in the middle, as shown in Figure 1.

Test facility

Figure 2 illustrates some views of the experimental system used in the compression tests of CFST columns under fire conditions, in which all the components are labeled. The experimental set-up comprised a two dimensional (2D) reaction steel frame (1) and a 3D restraining steel frame (2) adaptable for different levels of stiffness in order to simulate the axial and rotational restraint provided by a surrounding structure to a CFST column in fire. The 2D reaction frame was composed by two HEB500 columns with 6.6 m in height and by one HEB600 beam with 4.5 m in length using M24 steel grade 8.8 bolts in the connections. To achieve the desired levels of stiffness of the surrounding structure with the purpose of imposing axial and rotational restraint to the thermal elongation of the CFST columns, a 3D restraining frame with four HEB300 columns and four HEB400 beams (2 on the top and 2 on the bottom, arranged orthogonally) was assembled. Note that the stiffness could be modified every time the position of the columns or the height of the beams cross-section was changed. During these experimental tests, only the position of the columns was different. As stated before, two axial restraining values of the surrounding structure were used: 30 and 110 kN/mm, corresponding to a 6- and 3-m span of the 3D restraining frame's beams, respectively. This restraining system intended to reproduce the actual boundary conditions of a CFST column when inserted in a real building structure. The connections between the

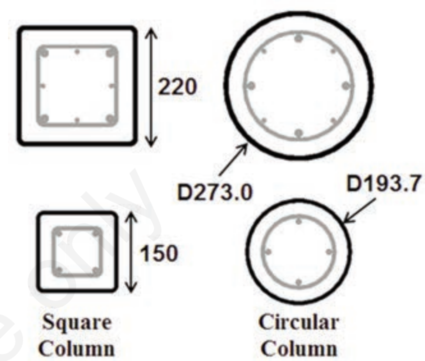

Figure 1. Scheme of the cross-sections of the tested columns.

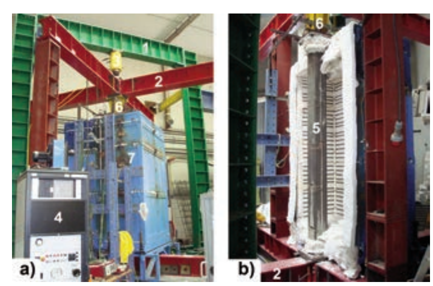

Figure 2. General (a) and detailed (b) view of the experimental set-up: 1, reaction frame; 2, restraining frame; 3, hydraulic jack; 4, central unit; 5, specimen; 6, load cell; 7, furnace.

Table 1. Test plan.

Test reference*	$\bar{\lambda}$	A_m/V (m^{-1})	$N_{b,Rd}$ (kN)	$k_{a,c}$ (kN/mm)	$k_{r,c}$ (kN.m/rad)	P_0 (kN)	k_a (kN/mm)	k_r (kN.m/rad)
CC273-30 k_a	0.41	14.7	3814	1094	24,695	1144	30	94,615
SC220-30 k_a	0.44	18.2	3751	1005	20,528	1125	30	94,615
CC194-30 k_a	0.58	20.7	1956	573	6632	587	30	94,615
SC150-30 k_a	0.65	26.7	1776	500	4820	533	30	94,615
CC273-110 k_a	0.41	14.7	3814	1094	24,695	1144	110	131,340
SC220-110 k_a	0.44	18.2	3751	1005	20,528	1125	110	131,340
CC194-110 k_a	0.58	20.7	1956	573	6632	587	110	131,340
SC150-110 k_a	0.65	26.7	1776	500	4820	533	110	131,340

$\bar{\lambda}$, non-dimensional slenderness of the column at ambient temperature; A_m/V, section factor; Nb,Rd, design buckling resistance of a compression member at ambient temperature according to EN 1994-1-1; $k_{a,c,}$ axial stiffness of the column; $k_{r,c,}$ rotational stiffness of the column; P_0, initial applied load on the column; k_a, axial restraint imposed by the surrounding structure; k_r, rotational restraint imposed by the surrounding structure; CC, circular column; SC, square column. *Number following CC or SC number indicates the diameter or edge length, respectively, while the last number indicates the axial restraint.

peripheral columns and the upper beams of the restraining frame were performed with M27 grade 8.8 threaded rods.

A hydraulic jack of 3MN capacity in compression (3) was hung on the 2D reaction frame (1) and controlled by a servo hydraulic central unit W+B NSPA700/DIG3000 (4) and, beneath this one, a load cell of 2 MN capacity in compression was mounted in order to monitor the applied load during all tests. The tested columns (5) were placed in the center of the 3D restraining frame and properly fitted to it (at each end plate) with four M24 steel grade 8.8 bolts, simulating semi-rigid ended support conditions. Additionally, above the specimen a 3MN compression load cell was mounted to monitor the axial restraining forces generated in the CFST columns during the whole test. Note that this load was inside a special device (6), which consisted of a hollow and stiff (massive) cylinder rigidly connected to the upper beams of the 3D restraining frame and to the top steel end plate of the specimen. The massive cylinder was placed inside the hollow cylinder and the load cell positioned between the massive cylinder and the steel end plate of the hollow cylinder. The lateral surface of the massive cylinder was Teflon lined so that it could slide through the hollow cylinder with very low friction. Regarding the thermal action, the specimens were heated by means of a vertical modular electric furnace (7), which was programmed to reproduce the standard fire curve ISO 834[19] (Figure 2).

Test method

The full-scale fire tests on CFST columns were conducted under transient state fire conditions. Hence, to achieve the goals of this investigation, these experimental tests were basically performed in two stages: loading and heating stage. The specimens were first axially loaded up to the target force under load control at a rate of 2.5 kN/s. The load level applied on the columns – P_0 – was 30% of the design value of the load bearing capacity of the columns at ambient temperature (Table 1), calculated in accordance with the methods proposed in EN 1994-1-1:2004[20] (*i.e.*, based on characteristic material properties). This loading intended to simulate a common serviceability load of a CFST column inserted in a real building structure. The HEB400 beams of the 3D restraining frame above the test specimen were, in a first step, not connected to the respective HEB300 columns with the purpose of applying the downward load on the test column. In other words, during the load application, the vertical displacements of these beams were allowed as a slide, guaranteeing vertical rigid body movement of the upper beams of the 3D restraining frame and thus ensuring that the defined axial load was directly applied to the test specimen. Then, when the load reached the desired level, the connections between the upper beams and the peripheral columns of the restraining frame were fastened with locking nuts, washers and threaded rods in order to simulate the axial and rotational restraint to the thermal elongation of the tested column under fire conditions.

Finally, at the second phase, the heating stage was started after the desired load had been reached and the vertical movement of translation of the upper beams of the 3D restraining frame was totally restrained. The ambient temperature at the start of the tests was about 20°C. During this stage the column was exposed to heating controlled in such a way that the average temperature in the furnace followed as closely as possible the ISO 834 standard fire curve.[19]

Throughout the tests, temperatures in the furnace and at several points of the columns, as well as end rotations, axial and horizontal displacements and axial compression forces (P) in the columns were measured to accomplish the objectives of this study and consequently to assess the fire resistance of the CFST columns.

Four linear variable displacement transducers (LVDT) were orthogonally arranged and positioned both on bottom and top of the steel end plates of the test columns to measure their axial displacement and rotations at their ends. The lateral displacements of the column were measured by three linear wire transducers (LWT) placed in two orthogonal directions at a 0.85, 1.90 and 2.56 m height in relation to the bottom of the column. Three type K (chromel-alumel) thermocouple probes were used to measure the furnace temperature along its height and above twenty type K wire thermocouples were used to measure the temperature in the specimens at five cross-sections and different depths, as can be seen in Figure 3.

During the heating period, the load was kept constant until the specimen failed. The failure criterion adopted in this paper was based on the buckling strength of the columns, in other words, a column was considered to have failed when the column could no longer have load-bearing capacity against the same amount of compression load, which had maintained before fire. Therefore, the critical time (fire resistance) in these tests corresponded to the time when the axial compression force in the column (measured value in the load cell placed between the specimen and the upper beams of the 3D restraining frame) returned to the value of the initial applied load (P/P_0=1), such as established in other research works which were focused on the fire behavior of members with restrained thermal elongation.[10-15]

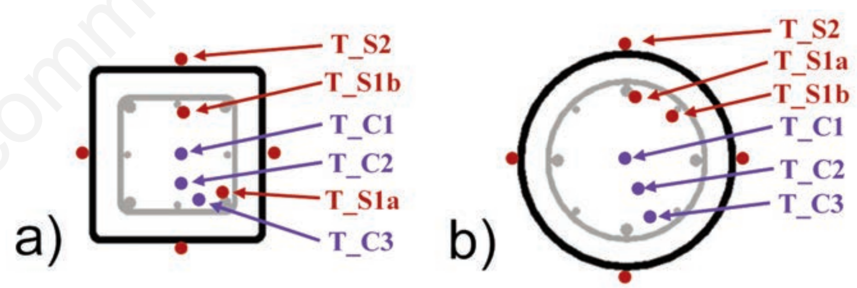

Figure 3. Location of the thermocouples in the square (a) and circular (b) columns.

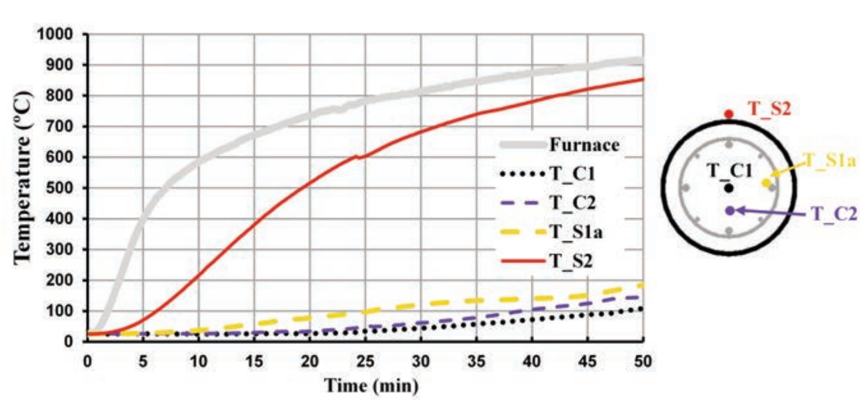

Figure 4. Evolution of temperature at mid-height of the tested column CC273-110 ka as a function of time.

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Results and Discussion

In Figure 4 it is presented, for instance, the evolution of temperatures in a circular cross-section at mid-height of the tested columns CC273-110 ka, as well as the respective furnace temperature. Although the furnace temperature was higher than 700°C for over 30 min, the temperature in the column core (T_C1) and on the longitudinal steel reinforcing bars (T_S1) did not exceed 150 and 200°C, respectively. This means that the buckling strength loss of the CFST columns was essentially due to deterioration of the mechanical properties of the steel tubular profile and of concrete between the tube and the rebars. From 30 minutes of test run, the temperature in steel tube approach the furnace temperature, with the difference between them being greater than 10% but less than 20%.

Each measuring point (such as T_C1, T_S1 and T_S2) was also used to establish the temperature profile in the vertical direction of the columns for different time instants during the tests, especially at their ending (Figure 5). The high thermal gradients near the ends of the columns were due to the fact that the first 300 mm from the bottom and top of the column were not directly exposed to the heating because these parts were covered by the furnace walls. Nevertheless, a large thermal gradient was observed from the surface to the core center of the CFST column (from T_S2 to T_C1), reaching temperature differences around 700°C. Note that great part of this gradient was concentrated near the surface of the column, where the difference between the thermocouples T_S2 and T_S1 at mid-height was about 618°C for the circular column (CC273-110ka) at the end of the test (Figure 5). The curves shown in Figure 6 are typical of the relative axial restraining force (P/P_0) and time curves for columns subjected to fire. As it can be seen, these curves had nearly the same trend but they may differ in the values and their rates depending on the studied parameters, especially on the slenderness and axial restraining level. The axial restraining forces

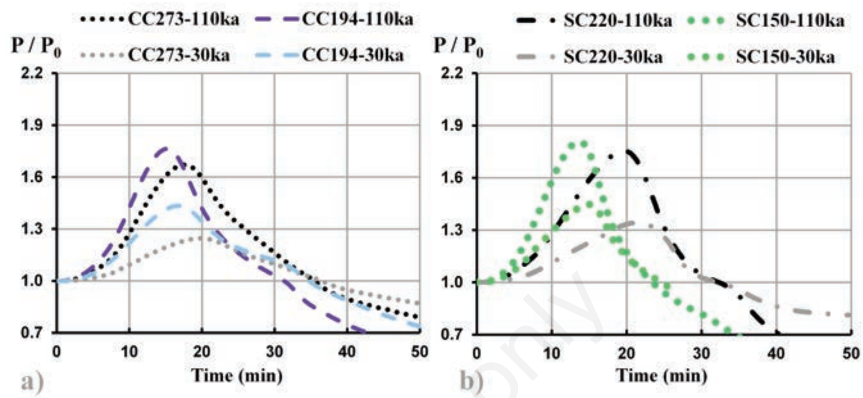

Figure 6. Evolution of the relative axial restraining forces generated in the circular (a) and square (b) columns as a function of time.

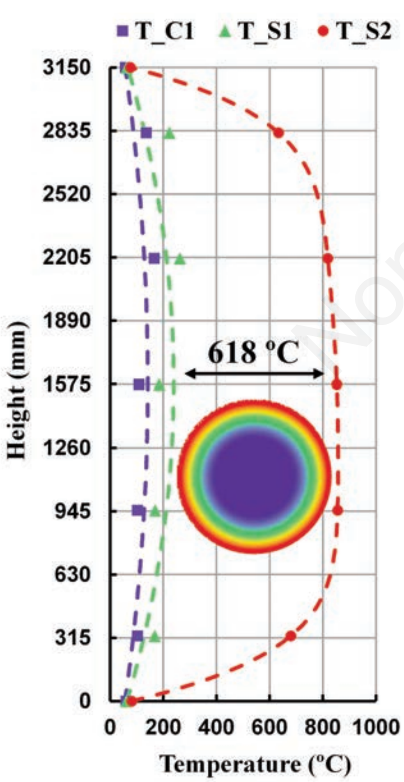

Figure 5. Temperature distribution along the height of the tested columns CC273-110 ka at the end of the test.

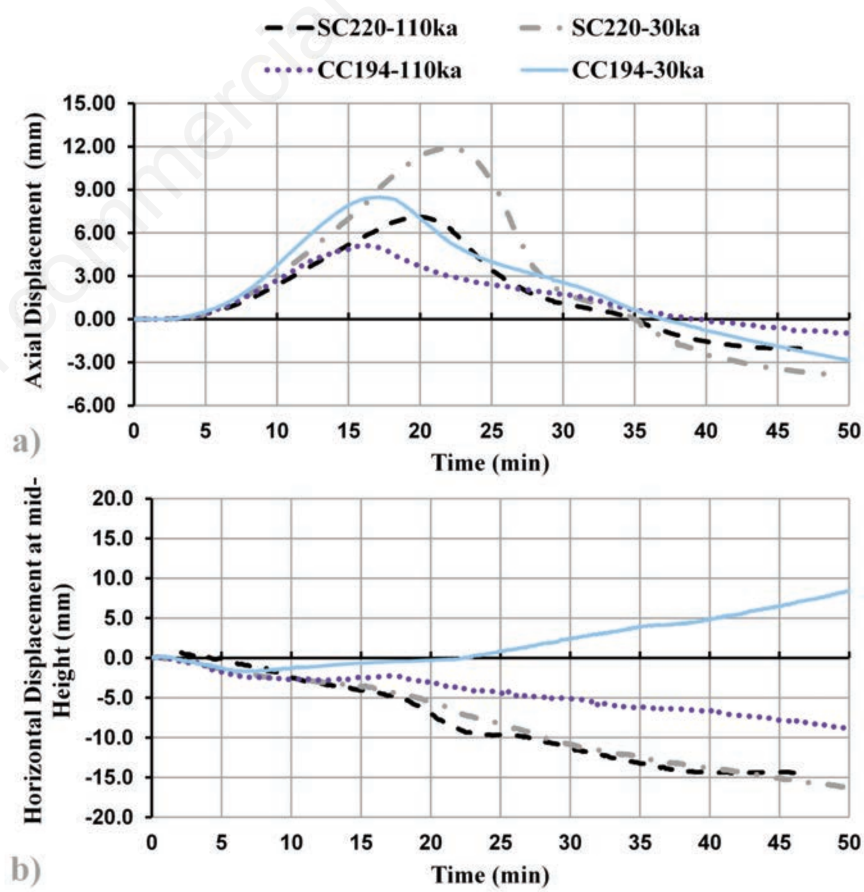

Figure 7. Evolution of the axial (a) and horizontal (b) displacements of columns (SC220 and CC194) as a function of time.

generated in the columns increased up to a maximum and then decreased up to failure mainly because of the loss of mechanical properties of the steel and concrete. These curves show clearly that the higher the stiffness of the surrounding structure, the higher the maximum axial restraining forces generated in the columns was. For instance, when the minimum axial restraint was used, the maximum axial compression force – P_{max} – in the square columns increased approximately 40% of the applied load, and 80% for the maximum axial restraint. It is quite interesting to observe that the failure in the square columns appeared quite early, compared to circular columns. The fire resistance of the column SC150-30 ka was about 25 min, whereas it was about 35 min for the column CC194-30 ka, corresponding to an increase of 40%.

Although it seems that the fire resistance of the CFST columns did not change with the stiffness of the surrounding structure (just about 5%), it is clear that the post-critical structural behavior of the columns depended significantly on the axial restraint level. In other words, after critical time, the axial restraining forces decreased faster for the columns under the maximum axial restraint than for the columns under the minimum axial restraint, as it can be seen in Figure 6. In addition, it is interesting to observe that the degradation stage of the buckling strength of the columns up to the critical time (*i.e.* the period of decrease in restraining forces) was slightly quicker than the loading stage due to the effect of the thermal action (*i.e.* the period of increase in restraining forces). In general, the maximum axial compression force in columns – P_{max} – was reached between 50 and 60% of the critical time. After this maximum, the compression forces began to decrease mainly as a result of the deterioration of the materials' mechanical properties with temperature, the loss of column stiffness, and the instability phenomenon. From the authors' point of view, the degradation of the buckling strength of CFST columns with restrained thermal elongation in a fire event can be divided into three stages. The first descending part of the axial restraining force and time curves (1st stage) may have been essentially due to the deterioration of the mechanical properties of the steel tube with temperature and due to the deterioration of the concrete's mechanical properties near the steel-concrete interface (concrete between the steel tube and the reinforcement steel bars) at the ending part of the curve (2nd stage). Lastly, after critical time (3rd stage), the column strength kept degrading as a consequence of the loss of the column resistance and stiffness, leading the columns to buckle.

From Figure 6, it is also possible to observe that despite the low fire resistance of the square columns, their resistance was still very prone to its slenderness, in contrast to the circular columns. When the non-dimensional slenderness increased from 0.44 (SC220) to 0.65 (SC150), the critical time decreased by 25% (from 32 to 24 min).

Figure 7 illustrates, as an example, the evolution of the axial (Figure 7a) and horizontal (Figure 7b) displacements in some CFST columns. As expected, the higher the axial stiffness of the surrounding structure, the lower the axial displacement of the columns was. The maximum axial displacement of the column SC220 was 11.9 mm for a 30 kN/mm axial restraint and 7.1 for a 110 kN/mm axial restraint, corresponding to a 40% decrease. However, identical columns started to buckle approximately at the same time (when their maximum axial displacement was reached). As shown in Figure 7b, the horizontal displacement rate of the columns at mid-height was significantly higher after its maximum axial displacement was reached than before. On the other hand, in Figure 8 the final lateral displacements along the major (a) and minor (b) axis are presented for the column SC220-110ka as example, in order to show their global deformed shape. By observing the obtained curves for the lateral displacements throughout the length of the columns it can be seen that the square columns buckled about both axes, with a 22.5 mm maximum horizontal displacement.

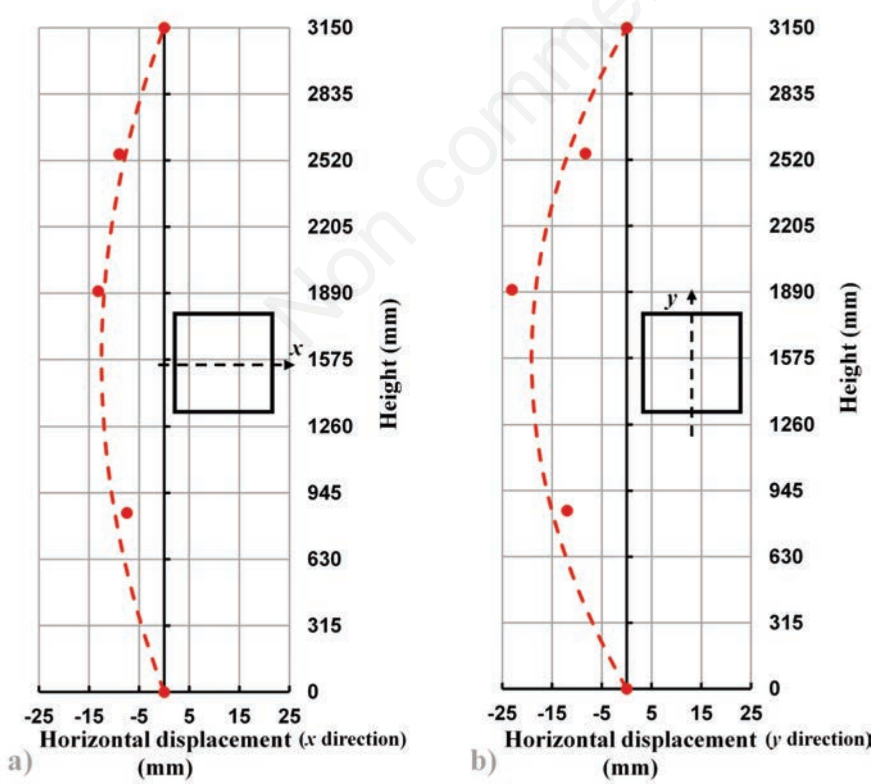

Figure 8. Horizontal displacements of the column SC220-110 ka at failure time (35 min) along the major (a) and minor (b) axis.

Conclusions

The results of this research study showed mainly that the fire resistance of identical semi-rigid ended columns may be not significantly affected by the stiffness of the surrounding structure, at least when there is significant rotational restraint (*i.e.*, when the level of rotational restraint on a CFST column is higher than 75%). When the axial and rotational restraint increased respectively from 30 to 110 kN/mm and from 94,615 to 131,340 kN.m/rad, the fire resistance of the CFST columns decreased by merely 5% on average. Note that the order of magnitude of axial restraint imposed to this type of members by the cold part of the surrounding structure in real buildings is usually of the order of magnitude of 2-3% of the member axial stiffness[21] and it is expected that the fire resistance of the columns remains constant for any degree of axial restraint higher than 10%.[22] All this might mean that the buckling length coefficient of a column on an intermediate floor of a building subject to fire might be higher than 0.5, and not equal to 0.5 as recommended by EN 1994-1-2:2005.[4]

In addition, the fire resistance of the CFST columns depended on the section shape, with the square columns being the less efficient section when exposed to fire. For the same load level (30%), the fire resistance ranged approximately from 23 (a square column) to 37 min (a circular column), corresponding to a 15 min difference.

On the other hand, the slenderness of the columns should be another parameter to take into account in their fire resistance. Although this one was less sensitive to the slenderness of the columns than to the section shape, the fire resistance of the square columns was still strongly dependent on their slenderness: when the non-dimensional slenderness increased from 0.44 to 0.65, their critical time decreased by 25%.

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