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BUCKLING OF CONCRETE FILLED STEEL HOLLOW COLUMNS IN CASE OF FIRE

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Abstract. *The use of concrete filled steel hollow sections (CSHS) in building construction has increased in the last decade. This type of columns has several advantages linked to their high load bearing capacity and good fire performance. However, when the columns are inserted in a building structure, the effect of the restraining to their thermal elongation, change significantly their fire behavior. The high restraining forces developed during the fire, associated to the degradation of the mechanical properties, will lead to their buckling sooner than they are isolated. In order to study these phenomena a series of fire resistance tests on CSHS columns was carried out in University of Coimbra. Several parameters expected to have influence in the fire resistance of these columns were studied: the slenderness of the column, the load level, the stiffness of the surrounding structure, the percentage of steel reinforcement and the thickness of the concrete layer inside the column. This paper presents some results of the fire resistance tests and analyzes the different buckling phenomena observed in the tested columns.*

1 INTRODUCTION

The use of concrete filled steel hollow sections (CSHS) in construction is increasing due to many advantages such us the high load-bearing capacity, possibility of use small cross-sections, short construction times, avoid the use of formworks in casting and enhanced fire performance.

The behavior in fire situation of CSHS columns has been studied by several authors for years [1, 2], but the majority of these studies do not consider the restraining to their thermal elongation. The behavior of CSHS columns inserted in a building structure and subjected to fire is different from when isolated. Thermal restraint promoted by the surrounding structure plays a key role in the stability of the column. There are many ways in which the column can interact with the surrounding structure, including the restraining to its thermal elongation, change of the column bending stiffness relative to the adjacent structure, and increasing of P-δ effect due to its lateral deformation. The surrounding structure induces additional axial forces and moments on the column those varies with temperature and are dependent of the degree of axial and rotational thermal restraint [3, 4].

In 2000, Rodrigues [5] published the results of a series of 168 fire resistance tests on compressed steel elements with restrained thermal elongation. Parameters such us the slenderness of the surrounding structure, eccentricity of the load, type of end supports and restraining stiffness, were tested. It was showed that for the case of pin-ended elements with centered loading as higher is the stiffness of the surrounding structure smaller is the critical temperature. The buckling of the elements with centered loading occurred suddenly while the ones with eccentric loading occurred in a very gentle way.

In 1999, Valente & Neves [6] presented a numerical research using the FEM program FINEFIRE, to analyze the influence of axial and rotational restraint on the critical temperature of steel columns. It is showed that the increasing on axial restraint diminishes, in general, while the rotational restraint increases the critical temperature of steel columns.

In 1999, Kodur presented the results of a series of 75 fire resistance tests in CSHS columns [1]. The results suggested that the fire resistance of CSHS columns is between 60 and 120min. Reinforced concrete and steel fiber reinforced concrete CSHS columns presented more than 180min of fire resistance. The failure mode of these columns was global buckling especially for the cross sections with diameter smaller than 203mm.

The major conclusions of the studies carried out on the area are that the load level, the cross-sectional dimensions, the effective length of the column, the slenderness ratio, the degree of concrete filling of the column, the type of filling (i.e. filled with concrete, reinforced concrete or fiber reinforced concrete) had a significantly influence in the fire resistance.

However, the influence of other parameters, such as the strength of the concrete and steel, the type of aggregate and the load eccentricity had a moderated influence, while the steel reinforcement ratios, the thickness of the steel tube's wall and the axis distance of reinforcing bars to this wall had a small influence on the fire resistance.

In order to study the influence of several parameters that may affect the behavior of CSHS columns subjected to fire, a set of fire resistance tests have been performed in the Laboratory of Testing Materials and Structures of the University of Coimbra. Parameters such as the slenderness of the column, the load level, the stiffness of the surrounding structure, the percentage of steel reinforcement and the thickness of the concrete layer inside the column, were tested. In this paper several results of these tests are presented and the main failure modes of the columns discussed.

2 EXPERIMENTAL PROGRAM

2.1 Test set-up

Figure 1 presents a general view of the test set-up built in the Laboratory of Testing Materials and Structures of the University of Coimbra for studying the behavior in case of fire of building columns with restrained thermal elongation.

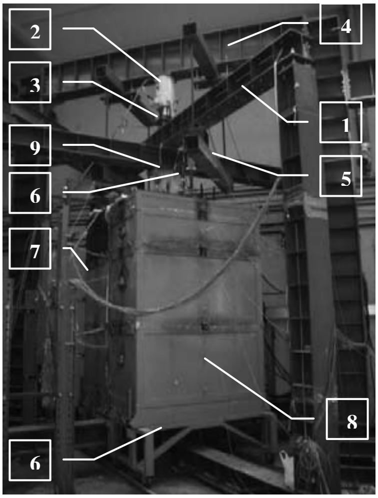

Figure 1. Test set-up

The system comprised a 3D restraining steel frame of variable stiffness (1) with the function of simulating the stiffness of the surrounding structure to the column subjected to fire. Several values of the stiffness of the surrounding structure were tested however in this paper will be presented only the results for 13kN/mm.

This restraining frame was composed by four columns, two upper beams and two lower beams, arranged orthogonally. The beams of this frame were made of steel profiles HEB300, grade S355. The connections between them were done by four M24 bolts, grade 8.8, except the connections between the columns and the upper beams that were used M27 threaded rods, grade 8.8.

The columns were subjected to a constant compressive load during the test that tried to simulate the serviceability load of the column when inserted in a real building structure. This load was a percentage of the design value of buckling load at room temperature calculated according to [7]. The threaded rods that connect the columns of the 3D restraining frame to its upper beams were initially completely loosened during the application of this load. This allowed the complete transfer of the load to the testing columns.

This load was applied by a hydraulic jack of 3MN (2) and controlled by a load cell (3) placed between the upper beam of the 3D restraining frame and the head of the piston of the hydraulic jack. The hydraulic jack was fixed in a 2D reaction frame (4), in which was also mounted a safety structure (5) to prevent damages in the experimental setup in case of sudden collapse of the column.

The axial displacements of the testing columns were measured by seven linear variable displacement transducers (LVDT), three in the top plate and four in the base plate (6). Their location is presented in figure 2.

Six cable LVDT (7) were used to measure the columns lateral deformation, in three different sections (S1, S2, and S3) and two orthogonal directions (figure 2).

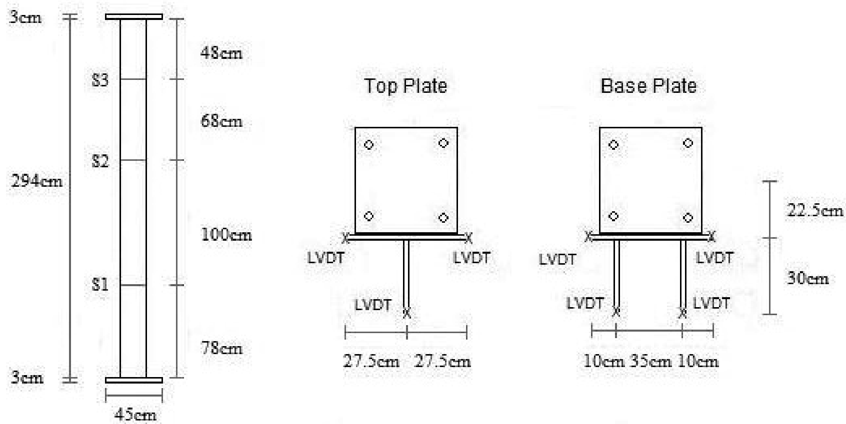

Figure 2. Position of LVDT in the testing columns

The thermal action was applied by a modular electric furnace (8) composed by two modules of 1m height and one module of 0.5m height, placed on top of each others, forming a chamber around the column of about 1.5m x 1.5m x 2.5m. There were three thermocouples type K, at middle height of each module, to measure the furnace temperatures.

A special device was built to measure the restraining forces generated by the testing column during the heating process (9). It consists of a hollow and stiff cylinder of high strength steel rigidly connected to an upper beam of the 3D restraining frame. On top of the testing column a massive steel cylinder was rigidly connected that entered into the hollow steel cylinder. The lateral surface of the massive cylinder was Teflon (PTFE) lined in order to avoid the friction between the cylinders.

The restraining forces were measured by a load cell of 3MN placed inside the hollow steel cylinder that was compressed by the massive cylinder due to the thermal elongation of the testing column during the fire resistance test.

2.2 Specimens

The steel profiles used in the fabrication of the columns had external diameter of 219.1mm and 168.3mm, both with 6mm of thickness and steel grade S355.

The concrete was class C20/25 [8] filling all the cross-section or forming a ring of 50mm for the greater column sections and 40mm for the smaller column sections around the tube internal wall.

The longitudinal reinforcement of the concrete had diameter of 12mm for the greater column sections and 10mm for the smaller column sections and steel grade A400. The stirrups used had 6mm diameter spaced each other 200mm. The concrete covering of the reinforcing bars was 30mm.

The temperatures in the columns were measured by thermocouples type K placed in five cross-sections along the height (S1 to S5) as showed in figure 3.

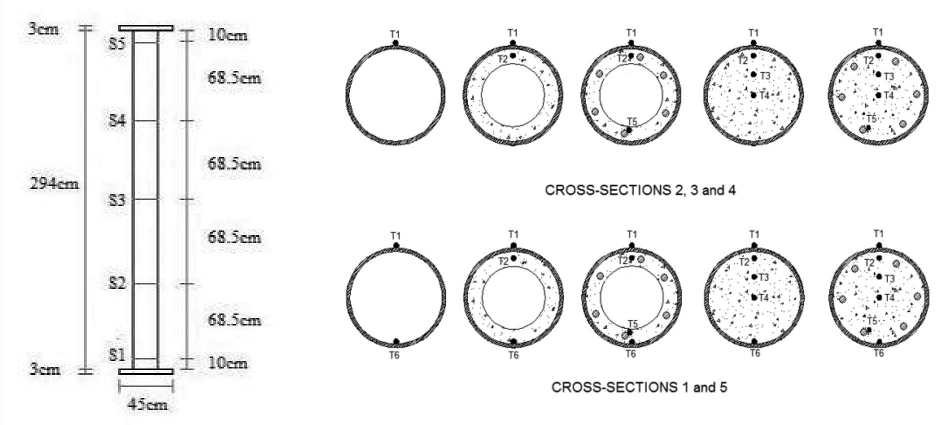

Figure 3. Position of thermocouples on test columns

2.3 Test plan

Table 1 presents part of the test plan already concluded and the correspondent failure modes of the test columns. The values of the serviceability load applied to the columns were 70 or 30% of the design value of the buckling load at room temperature ($N_{b,rd}$) [7].

The thermal action applied was the ISO 834.

Table 1: test plan and failure modes.

Column refer.	degree of concrete filling	cross-section (mm)	slenderness	reinforced concrete	$N_{b,rd}$ (kN)	Failure mode
A01	total	168.3	86.2	No	816 (70%)	global buckling
A03	total	168.3	86.2	No	350 (30%)	global buckling
A09	ring (40 mm)	168.3	78.5	No	775 (70%)	global/local buckling
A11	ring (40 mm)	168.3	78.5	No	332 (30%)	crushing
A13	--	168.3	63.1	--	589 (70%)	global/local buckling
A14	--	168.3	63.1	--	252 (30%)	global buckling
A15	total	219.1	66.2	No	1355 (70%)	global buckling
A17	total	219.1	66.2	Yes	1482 (70%)	global buckling
A18	total	219.1	66.2	No	581 (30%)	global buckling
A20	total	219.1	66.2	Yes	635 (30%)	global buckling

Table 1: test plan and failure modes (cont.).

Column refer.	degree of concrete filling	cross-section (mm)	slenderness	reinforced concrete	$N_{b,rd}$ (kN)	Failure mode
A21	ring (50 mm)	219.1	59.5	No	1239 (70%)	global buckling
A23	ring (50 mm)	219.1	59.5	Yes	1269 (70%)	global/local buckling
A24	ring (50 mm)	219.1	59.5	No	531 (30%)	global buckling
A26	ring (50 mm)	219.1	59.5	Yes	544 (30%)	crushing
A31	--	219.1	48.1	--	874 (70%)	global/local buckling
A32	--	219.1	48.1	--	375 (30%)	global buckling

3 RESULTS

3.1 Restraining forces

Figure 4 presents some results of the development of the restraining forces on the test columns in function of the mean steel temperature. The restraining forces, due to the restraining to thermal elongation, start to increase up to a maximum and then decrease due to the degradation of the mechanical properties. The test was ended when the restraining forces reached again the initial applied load. The deformed shape of the columns presented in figures of the next section corresponds, more or less, to that instant.

Concerning the type of failure of the columns it can be observed that A13 and A31 presented a sudden buckling. The restraining forces diminished quite suddenly probably due to the inexistence of concrete inside the tube. The columns with concrete inside presented a more gentle way of failure. The degree of concrete filling of the columns didn't lead to great changes in the mode of failure.

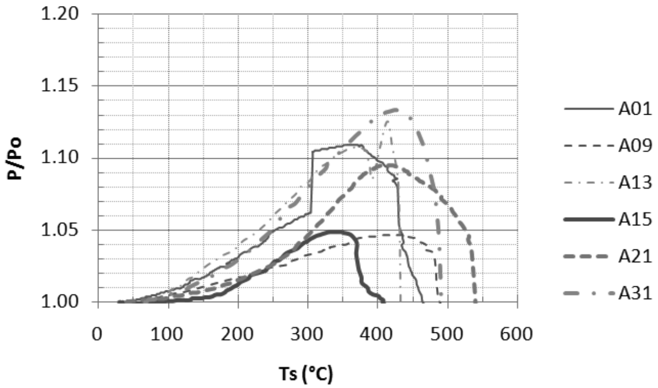

Figure 4. Restraining forces in function of the columns mean steel temperature

3.2 Failure modes

In general the failure mode of the columns was global buckling, in the major part of the cases, however local buckling also occurred in some cases (figure 6 - A09 and A13). The global buckling observed had a double curvature like a slight "s". As expected the failure mode is more clear in columns whose the applied load level was 70% $N_{b,rd}$ (figures 6 and 8).

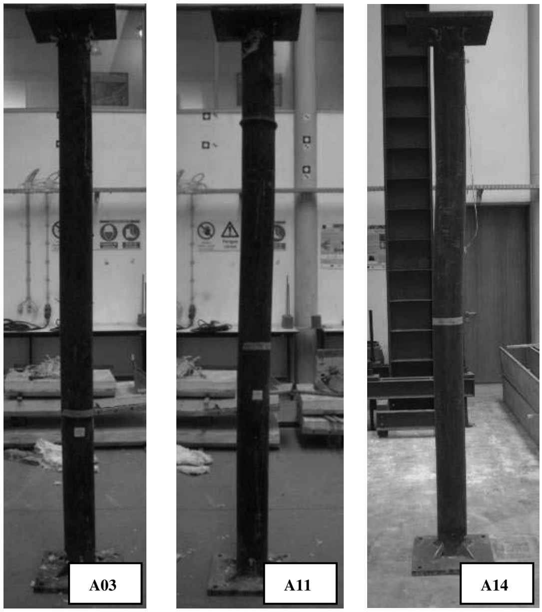

Figure 5. Columns totally filled with concrete (on left), with ring of concrete inside of 40mm (on center) and without concrete (on right), tube dia. = 168.3mm and load level =30% $N_{b,rd}$

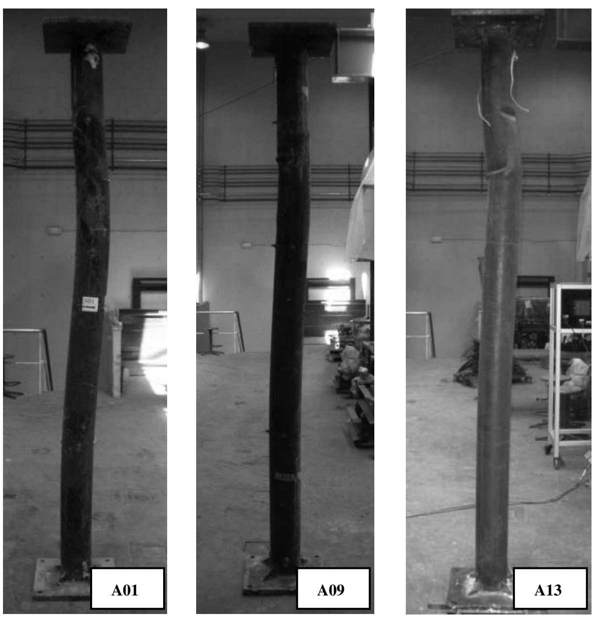

Figure 6. Columns totally filled by concrete (on left), with ring of concrete inside of 40mm (on center) and without concrete (on right), tube dia. = 168.3mm and load level=70% $N_{b,rd}$

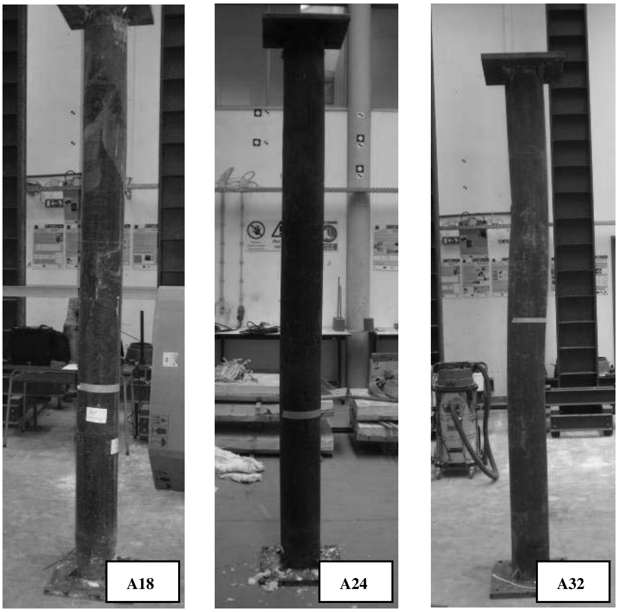

Figure 7. Columns totally filled by concrete (on left), with ring of concrete inside of 40mm (on center) and without concrete (on right)), tube dia. = 219.1mm and load level=30% $N_{b,rd}$

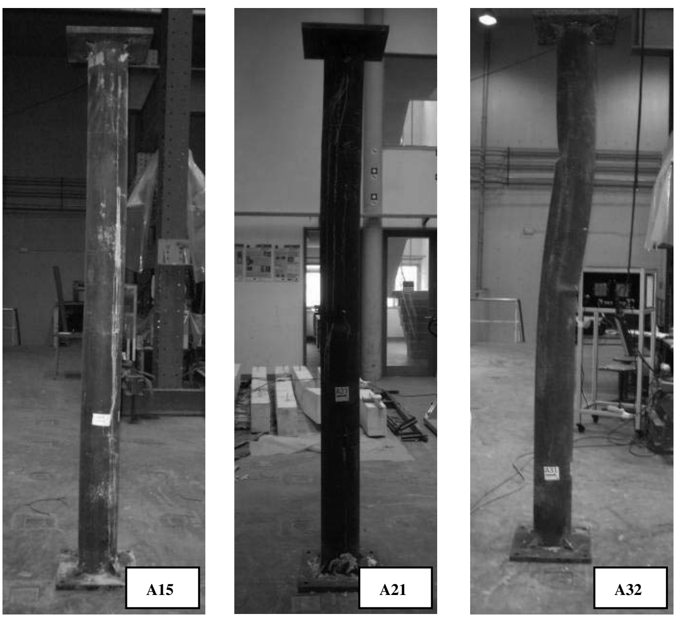

Figure 8. Columns totally filled by concrete (on left), with ring of concrete inside of 40mm (on center) and without concrete (on right)), tube dia. = 219.1mm and load level = 70% $N_{b,rd}$

Some columns presented failure by crushing (figure 5 – A11). For the load level of 70% $N_{b,rd}$ was not observed any failure by crushing.

The deformed shape of the columns was softer in the concrete filled columns than in bare steel columns. The concrete core remains colder than the steel profile in the fire event, avoiding local buckling and reducing the final deformed shape of the column. In this way the concrete inside the column enhances its behavior when subjected to fire. The local buckling was more evident in bare steel columns than in columns filled with concrete (figures 6 and 8).

The increase in columns cross-section reduced the slenderness of the column and the columns deformation shape after tests.

CONCLUSIONS

This paper reported the behavior of steel hollow columns, completely or partially filled with concrete, subjected to fire. They were presented the different modes of failure of these columns. Based on the results the following suggestions may be addressed.

- The load level of 70% $N_{b,rd}$ is too high for fire design of steel hollow columns and is advisable the use of a lower load level.
- The concrete inside the hollow column, especially when completely filing, it improves the fire behavior and reduces the possibility of local buckling.
- The reduction in the slenderness of the column enhances the behavior of the columns under fire.

This experimental study is part of a major research program that it is being carried out at the University of Coimbra. Others results and parameters that influence the fire resistance is also being investigated and will be presented in future.

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