12th International Conference on Advances in Steel-Concrete Composite Structures (ASCCS 2018)
Universitat Politècnica de València, València, Spain, June 27-29, 2018
Doi: http://dx.doi.org/10.4995/ASCCS2018.2018.7193

Analysis of concrete-filled steel tubular columns after fire exposure

C. Ibañez[a]*, L. Bisby[b], D. Rush[b], M. L. Romero[c] and A. Hospitaler[c]

[a]Department of Mechanical Engineering and Construction, Universitat Jaume I, Spain
[b]School of Engineering, University of Edinburgh, UK
[c]ICITECH, Universitat Politècnica de València, Spain
*corresponding author, e-mail address: ibanezc@uji.es

Abstract

Concrete filled steel tubular (CFST) columns have a high probability to resist high temperatures compared to steel structures, whose evaluation after a fire is limited by the resulting deformation. A better understanding of the behaviour of CFST columns after a fire, affected by the maximum temperature achieved by the concrete infill, is required to properly estimate their residual strength and stiffness in order to adopt a reasonable strategy with minimum post-fire repair. In this paper, a fiber beam model for the simulation of the post-fire response of slender concrete-filled steel tubular (CFST) columns is presented. First, the model is validated against experimental results and subsequently it is employed to analyse the post-fire response of circular CFST columns. The variation of the residual strength with the load level for realistic fire resistance times is numerically studied. Actually, in a building, the columns support load even while a fire is being extinguished, so it is important to take into account this loading condition when predicting the post-fire behaviour. Therefore, in this research, the complete analysis comprises three stages: heating, cooling and post-fire under sustained load conditions. The model considers realistic features typical from the fire response of CFST columns, such as the existence of a gap conductance at the steel-concrete interface or the sliding and separation between the steel tube and the concrete.

Keywords: *post-fire response; residual capacity; fiber beam model; concrete filled steel tubular columns.*

1. Introduction

In general, composite columns have a high probability to resist a fire compared to steel structures, whose evaluation after a fire is straightforward and limited by the resulting deformation. On the contrary, concrete-filled steel tubular (CFST) columns need a more detailed assessment since the concrete infill is deeply affected by the maximum temperature achieved during the fire. Therefore, a better understanding of the behaviour of CFST columns after fire is required to develop innovative techniques to estimate their residual strength and stiffness since it will allow adopting a reasonable strategy with minimum post-fire repair.

Regarding slender CFST columns, the number of works, both numerical and experimental is still scarce. However, some investigations on stub CFST specimens can be found which have served to assess the post-fire residual capacity of these composite columns. In this line, Han et al. [1] developed an experimental program on rectangular CFST columns. After exposure to the ISO834, with heating times of 90 min-180min, the columns were loaded up to failure at room temperature. A numerical model together with post-fire material models for stub CFST columns were also presented which are, to date, the only post-fire constitutive models proposed. According to the authors, slenderness ratio and fire duration have significant influence. Han and Huo [2] and Han et al. [3] continued this investigation.

As a novelty, Huo et al. [4] developed an experimental campaign on stub columns where

2018, Universitat Politècnica de València CC BY-NC-ND

the specimens were loaded during all the heating and cooling process. However, the finite element model presented by the authors was not able to model the cooling phase. It was concluded that pre-load has remarkable influence on the mechanical behavior of CFST columns, the failure of the fire-damaged column was still however ductile. The authors pointed out that the load and temperature had more influence in the stiffness of the columns than in the residual strength.

Rush et al. [5] presented a series of post-fire residual compression tests on CFST columns following the same procedure. Different to previous campaigns, some of the columns had higher relative slenderness, with a length of 1400mm. In this investigation, 19 tests on unprotected and protected CFST columns along with control tests on six unheated sections were tested. The specimens were exposed to fire and cooled to ambient temperature prior to be tested. With this research, the database of available tests including fiber reinforced and high strength concrete, protected specimens and, different heating regimes would be expanded. In addition, a detailed analysis of the cross-sectional temperatures was carried out [6].

Although employing cold-formed austenitic stainless steel hollow sections, Tao et al. [7] performed a series of tests on slender CFST columns in fire and after fire exposure. The columns were 1870 mm long but only the central 880 mm were exposed to a linear heating regime. The initial imperfection of the specimens and the deformation during the tests was monitored by photogrammetry to study the high sensitivity to initial imperfections of axially loaded slender columns under fire. A numerical model was also presented but only for the heating stage given the complication of implementing suitable models for the other stages, which is the main difficulty faced by researchers.

With regard to numerical models, it must be highlighted that the work of Yang et al. [8] who presented a FEM program for the entire fire exposure process under loaded conditions and the material properties developed by Han et al. [1] for stub columns were applied. Nevertheless, some realistic considerations were neglected in this model, such as the sliding between the steel tube and the concrete core, representative of the fire response of CFST columns.

This fact was also pointed out by Song et al. [9] who, aware of the importance of reproducing the interaction between the steel tube and the concrete core, presented a three-dimensional FEM to predict the fire behavior and residual capacity of stub CFST columns. Material models for each different situation were adopted as suggested in [8].

Yao and Hu [10] presented a FEM for the behavior of CFST columns after heating. Again, given the lack of experimental results, the FEM was validated against tests of stub CFST columns heated unloaded. It was found that steel recovers most of its strength and stiffness after cooling, but the concrete cannot recover when temperature exceeds 200ºC. The residual strength was strongly affected by the exposure time, the slenderness ratio and the cross-sectional diameter. A theoretical approach was proposed to calculate the residual strength of post –fire CFST columns.

A general review of the literature shows the lack of experimental and numerical studies for CFST columns, especially for slender members, which consider the whole fire process under constant loading condition.

In this work, a fiber beam model for the post-fire response of CFST columns is presented, which considers realistic features and is programed to reproduce the entire fire exposure process under constant load. The model is validated against experimental results and employed to carry out an analysis of the change in the residual strength with the load ratio for realistic fire resistance times.

2. Description of the model

The fiber beam model presented is based on the FedeasLab [11] platform, a Matlab toolbox for the nonlinear analysis of structures. It was initially developed and validated for the fire response of CFST columns [12] and now it is extended to simulate the post-fire response of these members. A brief description is given hereafter.

The model consists of three parts: the concrete core, the steel tube and the link elements, which connect the former two, as shown in Fig. 1. Therefore, a complete circular CFST column is formed by assembling in parallel the two components: the steel hollow section and the concrete core. These columns are

(cc) BY-NC-ND 2018, Universitat Politècnica de València

modeled with fiber finite elements connected at their nodes by link elements both longitudinally and transversely. The fiber beam-column element employed to model the two simple columns has a co-rotational formulation [13].

The interaction rules of the links connecting the elements obey both the room and the high temperature behavior. In order to assure identical deformed shapes of both components, the transversal link elements have a high stiffness. The marginal friction between the steel tube and the concrete core is reproduced by the inner longitudinal links. However, in order to reproduce the sliding that appears at high temperatures between the concrete core and the steel tube due to their different thermal expansion coefficients, the top longitudinal link shows an elevated stiffness under compression but zero stiffness under tension. The axial load is applied to the top node. In Fig. 1, the cross-sectional discretization pattern adopted after the calibration procedure is shown, which divides the section into a regular array of fibers in both radial and circumferential directions.

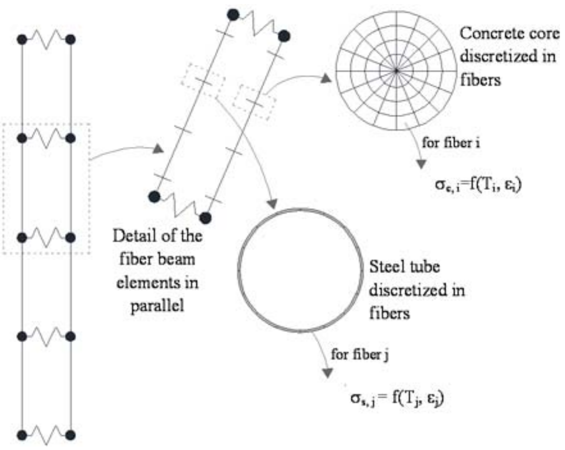


Fig. 1. Parallel model scheme and discretization of the section.

For the heat transfer problem, values from EN 1991-1-2 [14] are used. Gap conductance at steel-concrete interface is considered and the effect of concrete moisture is taken into account. For each type of material and at each stage, the corresponding thermal and mechanical properties should be adopted ([1], [8], [12]) following the approach of other authors [8, 9]. Nevertheless, the availability of suitable models in the literature is low, particularly for the cooling stage. Linear interpolation can be applied between two temperature dependent states to determine the state for a new temperature, but more research is needed to corroborate this assumption, as highlighted also Yang et al. [8].

3. Validation of the model

Given the lack of test data involving the sequence of stages of the whole fire exposure process (heating, cooling and post-fire stage under sustained load), the fiber model is validated separately against tests at different phases, including experiments on CFST at room temperature [15], subjected to fire [16, 17] and after fire exposure [5]. This approach was also successfully adopted by other authors when validating their proposed models [8, 9].

3.1. Room temperature response

The predicted load versus the deflection at the mid-height of the column is contrasted in Fig. 2 with the experimental results for one of the slender specimens used in validation. The detailed description of the reference tests, for both normal and high strength concrete infill, can be consulted in [15]. It can be seen that there is a good agreement between calculated and measured values.

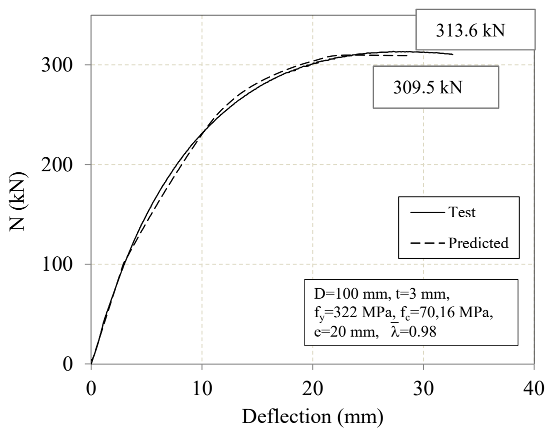


Fig. 2. Load-deflection curves at room temperature (C -100-5-2-60-20).

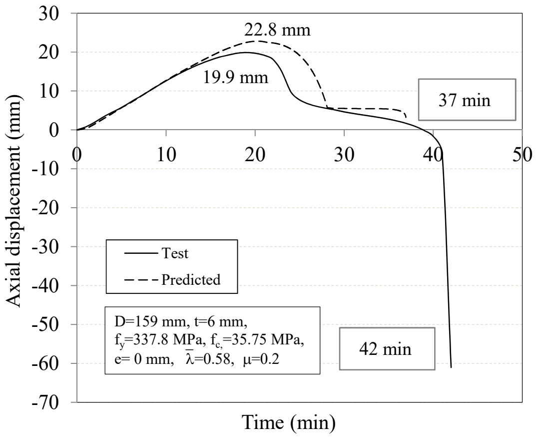
3.2. Fire response

The fire behavior of concentrically loaded CFST columns is simulated with accuracy by the model for different type of infills of both normal and high strength concrete. A sequentially coupled thermal-stress analysis is accomplished. This consists of two steps: (1) a sectional thermal analysis to compute the temperatures of the member and (2) a mechanical analysis. The heating regime curve is applied to the exposed surface along the whole length of the column. In this case, the confinement of the concrete is not

2018, Universitat Politècnica de València BY-NC-ND

taken into account in the mechanical analysis as a consequence of the existing sliding and separation between the two components produced by their different thermal expansion coefficients.

As an example, for two of the columns, the comparison of the fire response in terms of axial displacement-time is shown in Fig. 3. The description of these specimens and more details about validation can be found in [16, 17].



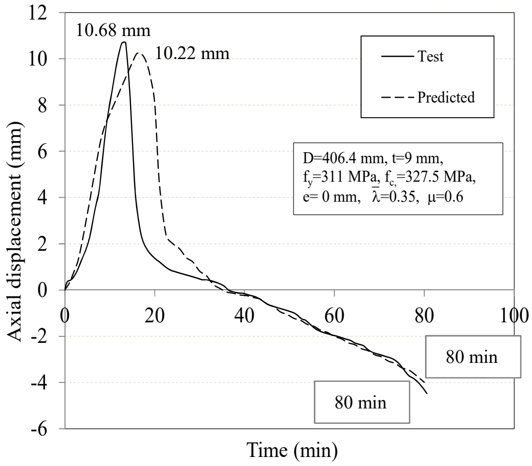


Fig. 3. Axial displacement – Time:
a) C-159-6-3-30-0-20; b) CBL-1.

3.3. Post-fire capacity

The third step is the validation against experimental data of the post-fire response at room temperature. As detailed above, most of the works reviewed investigate stub columns, but in the work carried out by Rush et al. [5], specimens with a relative slenderness of 0.6 and a length of 1400 mm were tested. However, it is important to note that further tests are required to provide experimental data for slender CFST columns.

Therefore, four specimens from this research were used in the validation and in the subsequent analysis. Table 1 summarises the details of the columns. All of them were filled with concrete of nominal compressive strength f_c=70MPa and the steel tubes had a yield strength of f_y=355MPa. The infill was fiber reinforced concrete for C1-C3 and plain for C4.

Table 1. Details of the columns [5].

Test	Name	D (mm)	t (mm)	Test N_{amb} (kN)	Test N_{post} (kN)
C1	C-10-F-I-N	139.7	10	1772	1061
C2	C-8-F-I-N	139.7	8	1664	813
C3	C-5-F-I-N	139.7	5	1372	591
C4	C-5-H-I-N	139.7	5	1346	583

The specimens were heated in a ceramic lined standard furnace for 120 min and cooled for at least 120 min in the furnace, recording the temperatures. All the heating process was carried out under unloaded conditions. In Fig. 4 the predicted and measured temperatures are plotted showing a good agreement. For the simulation, recommendations given in [6, 12] are adopted.

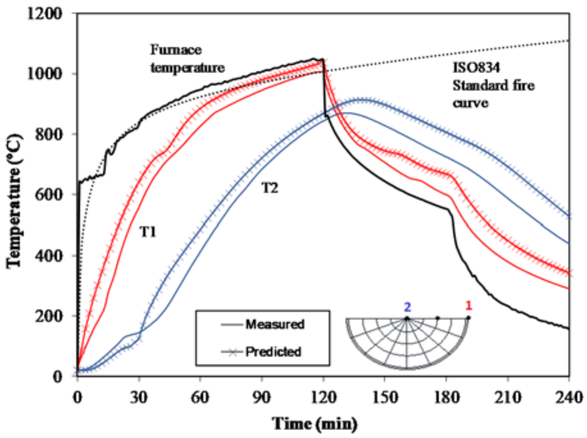


Fig. 4. Temperature-time for specimen C1.

The inertia existing in the concrete core is observed in the delay (of about 15 min) of the temperature curve, where the concrete temperature increases even after the furnace has started to cool down.

Finally, the specimens were tested as pinned-pinned up to failure. For each specimen, the load-deflection curve was registered during the test and the value of the residual strength (N_{post}) is extracted. Table 2 contains the value of the experimental and predicted residual strength

2018, Universitat Politècnica de València

values. Failure mode was global buckling and, in some cases, local buckling appeared.

In Fig. 5 the comparison between the experimental and the calculated curves is presented for two of the columns. As observed, the prediction of the load-deflection in the post-fire stage is reasonably good, with a mean error of 1.03, which lies on the safe side.

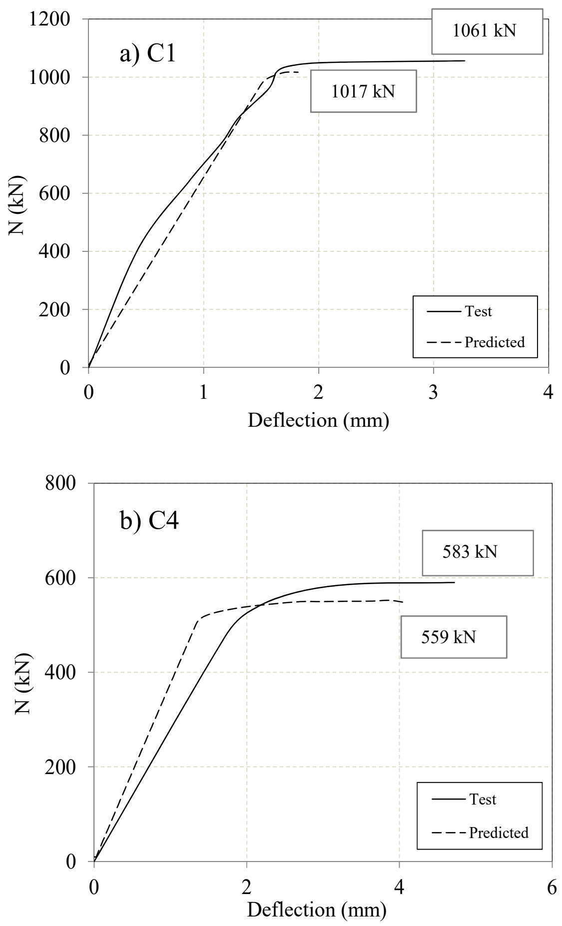


Fig. 5. Load-deflection curves at post-fire stage: a) C1 b) C4.

*Table 2.*Test and predicted values of the residual capacity.

Test	Test N_{post} (kN)	Predicted N_{post} (kN)	$\xi = \dfrac{N_{post,pred}}{N_{post,test}}$
C1	1061	1017	1.04
C2	813	849	0.96
C3	591	552	1.07
C4	583	559	1.04
		Mean	1.03
		SD	0.05

4. Analysis of the post-fire response under sustained load

4.1. Analysis procedure

The complete analysis comprises the three stages: heating, cooling and post-fire under sustained load. First, a constant initial load (N_o) is applied simultaneously with the beginning of the heating stage. The heating lasts until time t_h and, at this moment, the fire temperature starts to decrease and the cooling stage starts. Due to the thermal inertia, the descent of the concrete cross-sectional temperatures is delayed.

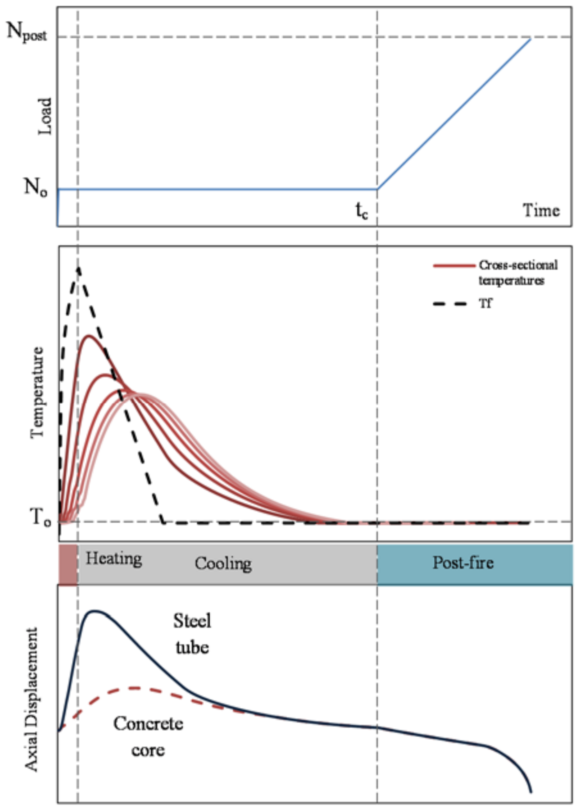


Fig. 6. Scheme of the analysis procedure.

The cooling stage continues until all the fibers in the section have reached the ambient temperature at time t_c. During these two stages the external applied load is maintained constant and equal to N_o. However, once the post-fire stage is reached, the load is increased by small increments up to failure. Fig. 6 shows schematically this process.

In this model, the standard ISO-834 [18] fire curve is applied for heating and cooling stages to the exposed surface of the column along its whole length and therefore a descending branch is considered according to EN 1991-1-2 [14].

2018, Universitat Politècnica de València CC BY-NC-ND

4.2. Influence of load and heating time on the residual capacity

In order to analyse the effect of the load ratio ($m=N_o/N_{ambient}$) and the heating time (t_h), new numerical analysis were carried out. The next study was accomplished by taking as reference the four specimens from Rush et al. [5] used for validation in the previous section, since their capacity at room temperature was known, and was used to calculate the residual strength index, which is given by *Eq. 1*:

$$\boldsymbol{r} = \frac{\boldsymbol{N}_{\text{post}}}{\boldsymbol{N}_{\text{ambient}}} \tag{1}$$

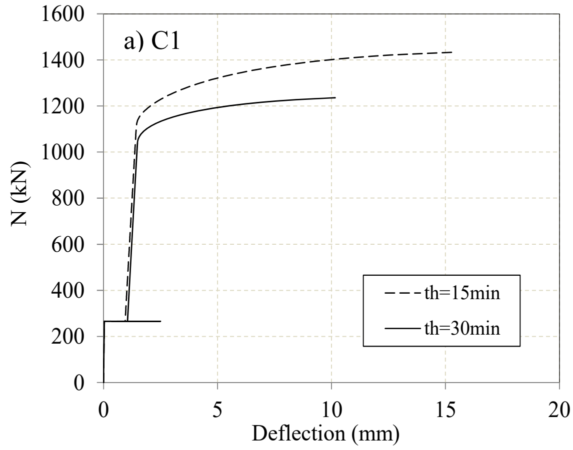
Each case of study is identified by its heating time and load ratio. Prior to the complete analysis, the fire resistance rating (*FRR)* values of all the columns were obtained in order to have a reference for the maximum heating time and establish, in consequence, the values of the different heating times (t_h). In this investigation, the maximum value of *FRR* reached was around 60 min. According to this maximum value and, in order to adopt a more realistic approach, standard fire resistance times were applied (i.e. R15, R30, etc.) since these are the actual values used in the practice. Considering all, the values adopted for the parameters in this analysis are those shown in Table 3.

Table 3. Variables values in the analysis.

Variable	Specified values
t_h (min)	15; 30
m	0.10; 0.15; 0.20

Therefore, 24 cases were analysed and the response during all the stages was registered. It must be noted that, although the range of variation of the load ratio was narrow, it was established to avoid the failure of the cooling during the cooling stage, so the whole analysis could be completed. According to the fire response predicted for this group of columns, higher values of m or t_h would cause the premature failure of the column. Nevertheless, as pointed out above, further tests and studies on this field are required for a better understanding.

In Fig. 7 the behaviour of cases C1 and C4 with load ratio m=0.15 is shown for the two heating times considered. The curve obtained for the load-deflection at mid-height was similar to that described by Yang et al. [8]. The plateau which interrupts the curve (Fig. 7) represents the effect of elevated temperatures at the heating and cooling stages. Given the non-uniform thermal expansion induced at the cross-section with the increase of temperature, redistribution of stresses takes place in order to maintain the planarity of the sections. As a consequence, the column deforms and deflection at mid-height augments. On the contrary, when the cooling stage starts, the column recovers to some extent its initial position. As observed in Fig.7, after the fire exposure, the residual lateral deflection value is around 1.5 mm. This value is in concordance to the results observed by Tao et al. [7] in their tests on slender stainless steel CFST columns, where the residual deflection ranged from 1 mm to 3 mm.



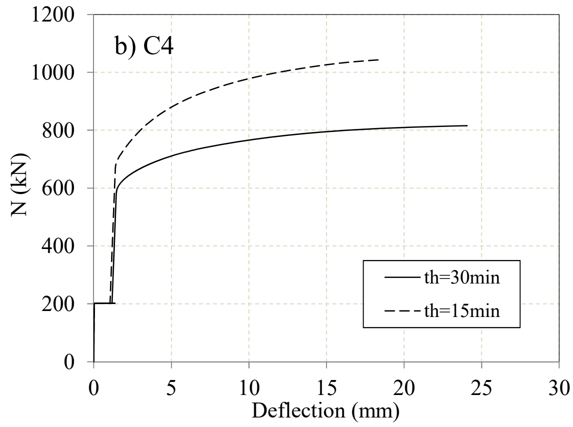


Fig. 7. Load-deflection curve for the complete analysis: a) C1 b) C4.

Finally, the effect of the load ratio and heating time on the residual capacity was evaluated. Again, in Fig. 8, for columns C1 and C4, the percentage of residual strength is plotted referenced to the load at ambient temperature of the columns. For clarity, it has been only plotted for m=0.10 and 0.20. It can be seen that, for the same load ratio, an increment in the heating time

(cc) BY-NC-ND 2018, Universitat Politècnica de València

results in a decrement of the post-fire capacity. However, the loss of strength with an increment in the load ratio is relatively small. This can be due to the fact that the columns slenderness is moderated and that the load ratios employed are still quite small to create a remarkable effect on the post-fire capacity. More studies covering a wider range of slenderness and load ratios would be necessary to completely understand the influence of the sustained load.

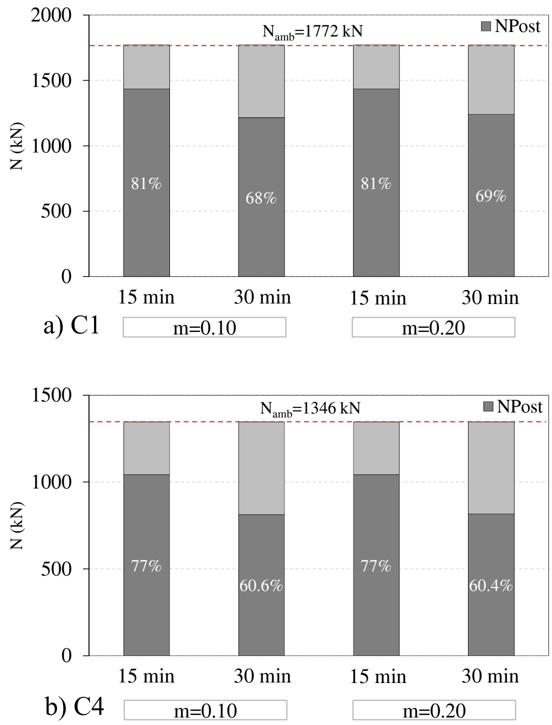


Fig. 8. Residual strength: a) C1; b) C4.

A complementary analysis of the data can be carried out comparing the non-dimensional values $N_{post}/N_{ambient}$ and t_h/FRR, given that the FRR of each column varies with the load ratio and, therefore, the same heating time can be more or less significant for a column according to its FRR.

Fig. 9 shows the variation of the residual strength index with the heating time for each value of the load ratio. For example, in case C1, if the column loaded with a load ratio m=0.20 is exposed to fire for t_h=15min, it implies a t_h/FRR=0.42 (42% of its FRR) but if it is loaded with a m=0.10, this heating time involves only a t_h/FRR=0.24 (24% of its FRR).

For case C1, the values of the residual strength index are, in general, higher than those of the rest of columns. This can be related to the fact that C1 is the specimen with the thickest steel tube, material which recovers practically its initial yield strength in the post-fire stage. For the rest of the cases, with thinner steel tubes, the values of the residual strength index decrease, reaching values around 65-60%. In addition, it can be confirmed the augment in the loss of strength with the increment in the heating time ratio.

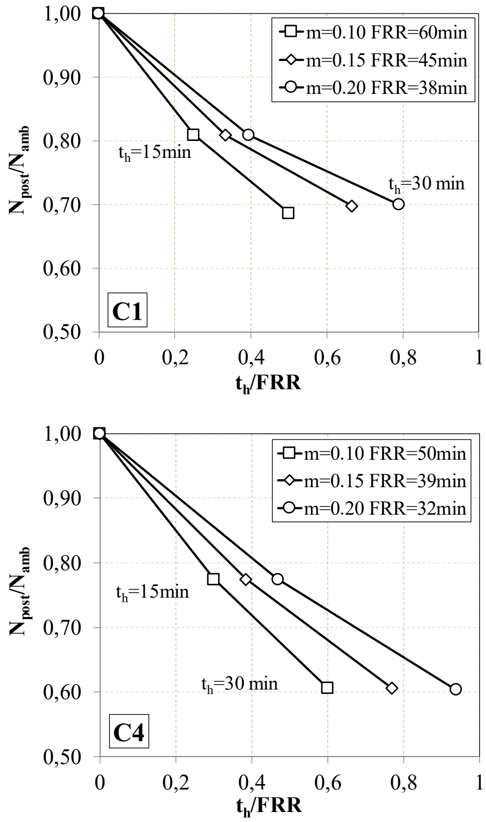


Fig. 9. Influence of load ratio on the residual strength index for columns a) C1; b) C4.

5. Conclusions

In order to avoid invasive techniques and minimise the post-fire repair, the proper interpretation of the post-fire response of CFST columns is essential to assess their residual strength. However, the number of related studies is limited, and specially the research concerning slender CFST columns. Besides, it is crucial to consider the loading condition of the column in the prediction of the post-fire response since during a real fire in a building, the columns are supporting load even during the fire extinguishing operations.

In this work a fiber beam model for the post-fire response of CFST columns was presented. The model considered realistic features and was designed to reproduce the response of an axially loaded CFST column during the three stages:

2018, Universitat Politècnica de València

heating, cooling and post-fire. The validation of the model with experimental results was presented separately for each type of response with considerable good agreement. The fire response of a series of columns reported in the literature was reproduced and analysed in order to investigate their response during a whole fire process subjected to load. Guidelines from others researchers regarding the material models have been followed, but the availability of suitable models, particularly for concrete during the cooling phase, is low.

Finally, the influence of the load ratio and the heating time on the residual strength of the columns was studied. The analysis of the results showed a high dependency of the heating time, which confirmed the conclusions drawn by other authors. More numerical and experimental studies are needed to completely understand the influence of the sustained load, especially for higher slenderness.

Acknowledgements

The authors gratefully acknowledge the financial support given by Generalitat Valenciana (Spain) for providing the funding BEST/2017/141 for the first author's stay as a visiting fellow at the School of Engineering of the University of Edinburgh.

References

[1] Han LH, Yang YF, Yang H, Huo J. Residual strength of concrete-filled RHS columns after exposure to the ISO-834 standard fire. Thin-Walled Structures 2002; 40: 991-1012.

[2] Han LH, Huo J. Concrete-Filled Hollow Structural Steel Columns after Exposure to ISO-834 Fire Standard. Journal of Structural Engineering 2003; 129 (1): 68-78.

[3] Han LH, Huo J, Wang YC. Compressive and flexural behavior of concrete filled steel tubes after exposure to standard fire. Journal of Constructional Steel Research 2005; 54: 882-901.

[4] Huo J, Huang G, Xiao Y. Effects of sustained axial load and cooling phase on post-fire behavior of concrete-filled steel tubular stub columns. Journal of Constructional Steel Research 2009; 65: 1664-1676.

[5] Rush DI, Bisby LA, Jowsey A, Lane B. Residual capacity of fire-exposed concrete-filled steel hollow section columns. Engineering Structures 2015; 100: 550-563.

[6] Rush D, Bisby L, Gillie M, Jowsey A, Lane B. Furnace tests on unprotected and protected concrete filled structural hollow sections. Fire Safety Journal 2015; 78: 71-84.

[7] Tao Z, Ghannam M, Song T-Y, Han LH. Experimental and numerical investigation of concrete-filled stainless steel columns exposed to fire. Journal of Constructional Steel Research 2016; 118: 120–134.

[8] Yang H, Han LH, Wang YC. Effects of heating and loading histories on post-fire cooling behaviour of concrete-filled steel tubular columns. Journal of Constructional Steel Research 2008; 54: 556-570.

[9] Song T-Y, Han LH, Yu HX. Concrete filled steel tube stub columns under combined temperature and loading. Journal of Constructional Steel Research 2010; 66: 369-384.

[10] Yao Y, Hu XX. Cooling behavior and residual strength of post-fire concrete filled steel tubular columns. Journal of Constructional Steel Research 2015; 112: 282-292.

[11] Filippou FC, Constantinides M. FEDEASLab. Getting Started Guide and Simulation Examples. Technical Report NEESgrid-2004-22, Berkeley, CA: Civil and Environmental Eng. Dept. University of California at Berkeley, 2004.

[12] Ibañez C, Romero ML, Hospitaler A. Fiber beam model for fire response simulation of axially loaded concrete filled tubular columns. Engineering Structures 2013; 56: 182-193.

[13] Spacone E, Ciampi V, Filippou FC. Mixed formulation of nonlinear beam finite-element. Computers & Structures 1996; 58 (1): 71-83.

[14] CEN. EN 1991-1-2, Eurocode 1: Actions on structures, Part 1.2: General actions - Actions on structures exposed to fire. Brussels, Belgium: Comité Européen de Normalisation; 2002.

[15] Portolés JM, Romero ML, Bonet JL, Filippou FC. Experimental study of high strength concrete-filled circular tubular columns under eccentric loading. Journal of Constructional Steel Research 2011; 64 (4): 623-633.

[16] Romero ML, Moliner V, Espinos A, Ibañez C, Hospitaler A. Fire behavior of axially loaded slender high strength concrete-filled tubular columns. Journal of Constructional Steel Research, 2011; 67: 1953-1965.

[17] Kim DK, Choi SM, Kim JH, Chung KS, Park SH. Experimental study on fire resistance of concrete-filled steel tube column under constant axial loads. Steel Structures 2005; 5: 305-313.

[18] ISO (International Standards Organization). ISO 834: Fire resistance tests, elements of building construction. Switzerland: International Standards Organisation; 1980.

(cc) BY-NC-ND 2018, Universitat Politècnica de València