Buckling behavior of cold-formed steel columns at both ambient temperature and simulated fire conditions

Hélder D. Craveiro, João Paulo C. Rodrigues, Luís M. Laím

Institute for Sustainability and Innovation in Structural Engineering, University of Coimbra, Coimbra, Portugal

Abstract

Cold-formed steel (CFS) profiles with a wide range of cross-section shapes are commonly used in building construction industry. Nowadays several cross-sections can be built using the available standard single sections (C, U, Σ, *etc*.), namely open built-up and closed built-up cross-sections. This paper reports an extensive experimental investigation on the behavior of single and built-up cold-formed steel columns at both ambient and simulated fire conditions considering the effect of restraint to thermal elongation. The buckling behavior, ultimate loads and failure modes, of different types of CFS columns at both ambient and simulated fire conditions with restraint to thermal elongation, are presented and compared. Regarding the buckling tests at ambient temperature it was observed that the use of built-up cross-sections ensures significantly higher values of buckling loads. Especially for the built-up cross-sections the failure modes were characterized by the interaction of individual buckling modes, namely flexural about the minor axis, distortional and local buckling. Regarding the fire tests, it is clear that the same levels of restraint used in the experimental investigation induce different rates in the generated restraining forces due to thermal elongation of the columns. Another conclusion that can be drawn from the results is that by increasing the level of restraint to thermal elongation the failure of the columns is controlled by the generated restraining forces, whereas for lower levels of restraint the temperature plays a more important role. Hence, higher levels of imposed restraint to thermal elongation will lead to higher values of generated restraining forces and eventually to lower values of critical temperature and time.

Introduction

There have been some significant developments in cold-formed steel (CFS) structures over the past few decades, mainly due to improving technology of manufacture (higher quality steels, more complex section shapes, improved forming technology) and corrosion protection. This leads to greater competitiveness of this structural solution that has been translated into an increasing market share throughout the world. In the past few decades' researchers have been focused on the behaviour of cold-formed steel structures. Regarding the behaviour of CFS columns, research has been mainly focused on open sections, such as plain and lipped channels, channels with simple and complex edge stiffeners, with and without holes, and angles.[1-4] More recently built-up members have also been investigated by some researchers. Built-up cross-sections present several advantages when compared with single sections. A built-up section can span more distance, present a higher load bearing capacity and higher torsional stiffness.[4] Also the use of built-up cross-sections can be a major economic advantage since all manufacture process remains the same.[5] Usually this type of cross-sections is built using self-drilling screws or seam weld.[6] At ambient temperature some research concerning the ultimate load-carrying capacity of built-up closed CFS columns has been conducted.[7]

However, so far, design methodologies presented in current design codes are still poor especially for built-up columns under fire situation. Traditionally two design methods are used, the Effective Width Method (EWM) used globally and the Direct Strength Method (DSM)[8] used in North America, Australia/New Zealand. The EN 1993-1-3:2006[9] only predicts that the buckling resistance of a closed built-up cross-section should be determined using the buckling curve b in association with the basic yield strength f_{yb}, and buckling curve c in association with the average yield strength f_{ya} provided that $A_{eff} = A_g$.

Regarding fire design the methods presented in the EN 1993-1-2:2005[10] for hot-rolled steel members are also applicable to cold-formed steel members with class 4 cross-sections. Also the EN 1993-1-2:2005[10] for class 4 cross-sections limits the critical temperature to 350ºC. Regarding the fire behaviour of cold-formed steel columns considering the influence of restraint to thermal elongation the investigations is still scarce. In a real building in case of fire it is most likely that a column may be subjected to thermal elongation. Since that column is restrained by the surrounding structure this particular column will be subjected to additional compressive forces that are generated due to the fact that the free thermal elongation is restrained by the surrounding structure. This phenomenon may lead to premature failure. Hence, it is clear that the effect of restraint to thermal elongation in cold-formed steel columns should be thoroughly studied.

To sum up, an extensive experimental investigation on the buckling behaviour of CFS columns at both ambient and simulated fire conditions was undertaken. Four cross-section shapes, two end-support conditions were tested in ambient temperature buckling tests. In the fire tests the same cross-section shapes were tested as well as two levels of service load (30 and 50% $N_{b,Rd}$) and restraint to thermal elongation.

Correspondence: Hélder D. Craveiro, Institute for Sustainability and Innovation in Structural Engineering, University of Coimbra, rua Luís Reis Santos, 3030788 Coimbra, Portugal.
Tel: +351.912168154 - Fax: +351.239797123.
E-mail: heldercraveiro.eng@gmail.com

Key words: Built-up; Cold-formed steel; Column; Buckling; Experimental.

Acknowledgements: the authors gratefully acknowledge the Portuguese Foundation for Science and Technology-FCT (www.fct.mctes.pt) and the cold-formed steel profile maker Perfisa (www.perfisa.net) for their support under the framework of the research project PTDC/ECM/116859/2010, as well as to the Human Potential Operational Programme (POPH), the European social fund and national strategic reference framework (QREN). This work was also supported by FCT, within ISISE, project UID/ECI/04029/2013.

Received for publication: 17 April 2016.
Accepted for publication: 15 November 2016.

Fire Research 2017; 1:16
doi:10.4081/fire.2017.16

Materials and Methods

Test specimens

All specimens consisted of one or more CFS profiles, namely plain channels (U) and lipped channel (C) profiles (Figure 1). All profiles were fabricated with S280GD+Z structural steel, hot dip galvanized with zinc on each side, and with yield strength of 280 MPa and an ultimate tensile strength of 360 MPa. Using the single sections it was possible to combine them to fabricate built-up cross-sections using self-drilling screws Hilti S-6.3×19MD03Z. The length of all profiles was 2950 mm and the spacing of the fasteners along the length of the column was 725 mm. In both bukling tests and fire tests each cross-section tested was instrumented with strain gauges and thermocouples, respectively. In the buckling tests the strain gauges were placed in several points of the cross-section at mid-height of each CFS column tested. In the fire tests thermocouples were positioned in different points of the cross-section and in five different sections along the length of the column.

Test set-up for buckling tests

The test set-up specifically designed for conducting buckling tests on CFS columns is thoroughly described in this section. With this experimental system it was attempted to simulate both pin and fixed-ended conditions in order to assess lower and upper bounds of the buckling load of the tested CFS columns. The experimental test set-up comprised a 2D reaction steel frame (i), a concrete footing (ii), the designed end-support devices (iii), load cell used to measure the applied load (iv), the hydraulic jack (v) used to apply the load to the CFS column, the servo hydraulic central unit W+B NSPA700/DIG2000 (vi) and the data acquisition system TML TDS-530 (vii) (Figure 2).

Test set-up for fire tests with restraint to thermal elongation

The experimental set-up comprised a two dimensional (2D) reaction steel frame (i in Figure 3a and b) and a three dimensional (3D) restraining steel frame adaptable for different levels of stiffness (ii in Figure 3a and b) in order to simulate the axial and rotational restraint imposed by the surrounding structure to a CFS column (vi) in fire. To achieve the desired levels of stiffness of the surrounding structure, in order to provide axial and rotational restraint (K1 to K4) to the thermal elongation of the CFS columns, two different 3D restraining frames were used in the experimental tests. In order to confirm the levels of stiffness, in addition to the experimental tests, some numerical simulations were carried out. Replacing the CFS column by a hydraulic jack a constant load was applied to the restraining frame and the respective vertical nodal (point of intersection of the top beams of the 3D restraining frame) displacement measured. The obtained values were also confirmed with the values of the restraining forces and axial displacements registered in the fire resistance tests of CFS columns. The rotational stiffness of both restraining frames (RF.1 and RF.2) was determined through numerical simulations using the finite element software Abaqus. From calculations and for the restraining frame with axial stiffness of 3 kN/mm a rotational stiffness of 9253 kN.m/rad and 2196 kN.m/rad was obtained. For the restraining frame with an axial stiffness of 13 kN/mm a rotational stiffness of 37237 kN.m/rad and 12620 kN.m/rad was obtained, respectively about the minor and major axis of the CFS column.

The connections between the peripheral columns and the top beams of the restraining frame were made with threaded rods. A hydraulic jack, placed in the 2D reaction frame, was used to apply the serviceability load (iii in Figure 3a and b). The thermal action was applied by a vertical modular electric fur-

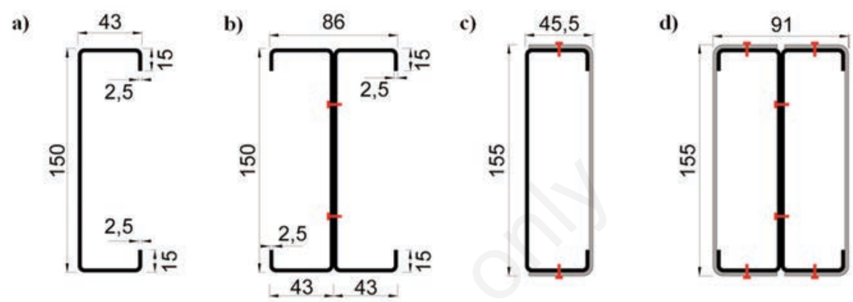

Figure 1. Dimensions of the cross-sections tested: a) C; b) I; c) R; d) 2R.

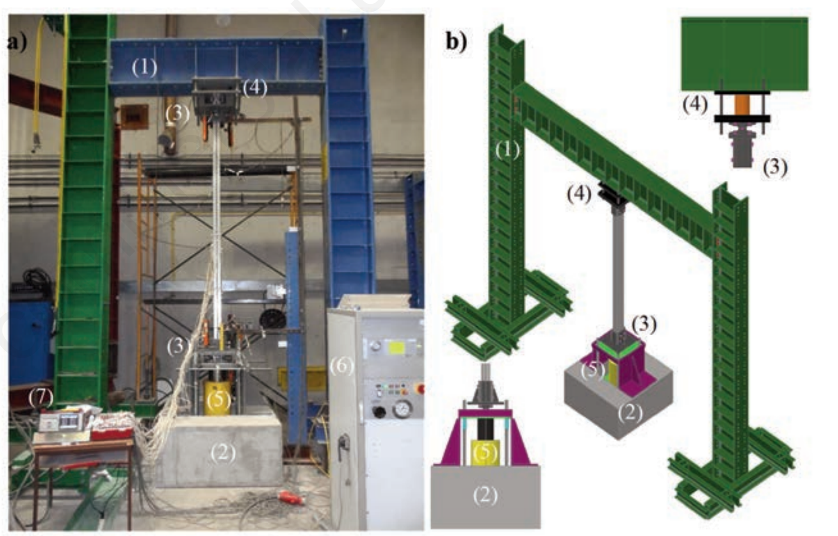

Figure 2. Experimental test set-up designed and built for the buckling tests. a) Test set-up; b) schema of the test set-up.

Table 1. Test programme and predicted values for cold-formed steel columns.

Test reference	$\bar{\lambda}$	$N_{b,Rd}$ (kN)	Load	End-support	Repetitions
C_PP_i	2.11	24.8	Until failure	Pinned	3
I_PP_i	1.62	85.51			3
R_PP_i	1.66	76.55			3
2R_PP_i	1.03	305.57			3
C_FF_i	1.51	41.80		Fixed	3
I_FF_i	0.9	187.70			3
R_FF_i	0.94	168.27			3
2R_FF_i	0.59	443.42			3

nace (iv in Figure 3b) programmed to reproduce the standard fire curve ISO 834. Both pinned and semi-rigid supports (vii) were adopted in this investigation. To measure the restraining forces generated on the testing column during the heating process a special device was built (v in Figures 3a and 4), consisting on a hollow steel cylinder where a stiff steel cylinder Teflon (PTFE) lined slides through it (Figure 4). On the top of the stiff steel cylinder a 500 kN load cell was placed and compressed against the top end plate of the hollow steel cylinder.

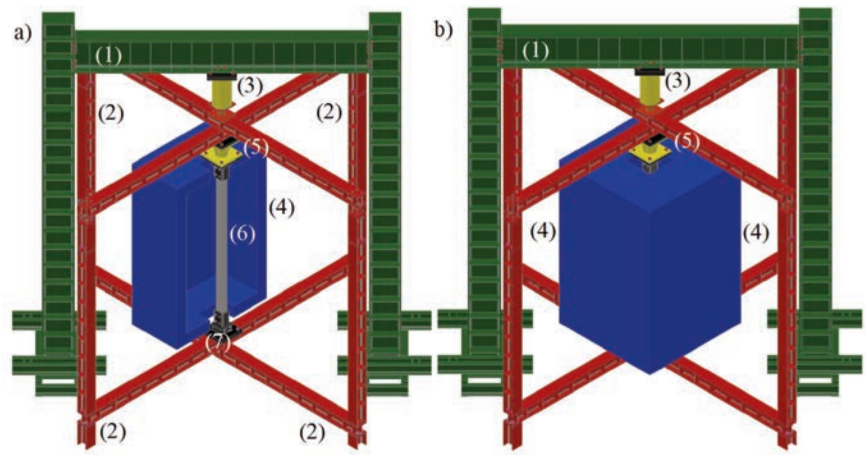

Figure 3. Schematic view of the experimental set-up.

Test plan

Buckling tests at ambient temperature

The experimental campaign undertaken to assess the buckling behaviour and failure modes of CFS columns consisted of 24 quase-static compression tests. In Table 1 the tests programme is detailed. The reference C_PP_1 indicates the first test (i) of columns with lipped channel cross-section (C) and with pinned-end support condition (PP), while the reference R_FF_3 indicates the third (iii) test of columns with closed built-up cross-section (2R) and with fixed end-support condition (FF).

Fire tests with restraint to thermal elongation

The experimental campaign on CFS columns under fire situation with restraint to thermal elongation consisted of 96 fire tests. Several parameters were assessed in this extensive experimental investigation, namely the cross-section shape, the influence of end-support conditions, influence of initial service load applied to the column and the influence of the level of restraint to thermal elongation imposed by the surrounding structure to the

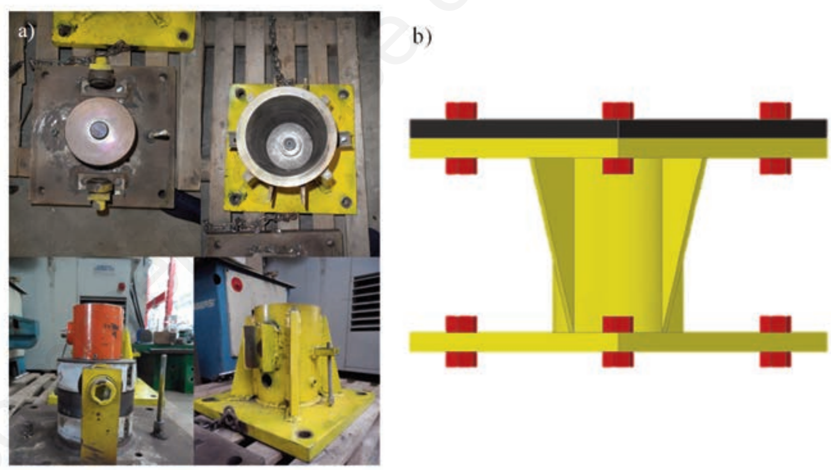

Figure 4. Device for measuring the restraining forces.

Table 2. Test programme and predicted values for cold-formed steel columns with closed built-up cross-section.

Test reference	$\bar{\lambda}$	$N_{b,Rd}$ (kN)	P_0 (kN) 30%	P_0 (kN) 50%	$K_{a,s}$ (kN/mm)	$K_{a,c}$ (kN/mm)
C_PP_*i*LL_K*j*	2.11	24.8	7.4	12.4	3 and 13	44.8
I_PP_*i*LL_K*j*	1.62	85.51	25.7	42.8		90.6
R_PP_*i*LL_K*j*	1.66	76.55	22.9	38.2		87.0
2R_PP_*i*LL_K*j*	1.03	305.57	91.6	152.7		175.4
C_SR_*i*LL_K*j*	1.51	41.80	12.5	20.9		44.8
I_SR_*i*LL_K*j*	0.9	187.70	56.3	93.8		90.6
R_SR_*i*LL_K*j*	0.94	168.27	50.4	84.1		87.0
2R_SR_*i*LL_K*j*	0.59	443.42	133.0	221.7		175.4

i, load level adopteed (30 or 50% Nb,Rd); j, level of restraint to thermal elogation imposed by the surrounding structure (K1 to K4).

CFS column. For each test condition three repetitions were conducted. For instance, lipped channel columns were tested considering two end-support conditions, two levels of service load and two levels of restraint to thermal elongation. Hence, 24 fire tests were undertaken for columns with lipped channel cross-section.

The applied load (P_0) corresponded to 30 and 50% of the design buckling load at ambient temperature ($N_{b,Rd}$). The design buckling loads were determined according to the provisions presented in EN 1993-1-1:2005,[11] EN 1993-1-3:2006[9] and EN 1993-1-5:2006.[12] The non-dimensional slenderness was determined according the provisions presented in the EN 1993-1-1:2005[11] for class 4 cross sections. The level of axial restraint imposed to a CFS column is defined as the ratio (eq. 1) between the axial stiffness of the surrounding structure to the CFS column ($K_{a,s}$) and the axial stiffness of the column ($K_{a,c}$) (eq. 1):

$$\alpha_{K.20^{\circ}C} = \frac{K_{a.s}}{K_{a.c}}$$

where $K_{a.c} = A_c E_c / L_c$. In Table 2, the reference C_PP_30LL_K1-2, indicates the second test (2) of the column with lipped channel (C) cross-section tested with 3 kN/mm of axial stiffness of the surrounding structure (K1) and a 30% load level (30LL) for the pin-ended support condition (PP), while the reference I_SR_50LL_K4-3 indicates the third test (3) of the column with built-up I (I) cross-section with 13 kN/mm of axial stiffness and 37237 and 12620 kN.m/rad of rotational stiffness about the minor and major axis of the CFS column (K4), respectively and a 50% load level (50LL) for the semi-rigid support condition (SR).

Results and Discussion

Buckling tests at ambient temperature

Figure 5 shows the obtained results in the experimental tests for both pinned and fixed-end support conditions. Load *vs* axial displacement curves are presented. It was observed that for the three tests conducted for each type of cross-section the loading stage, failure load and unloading stage was very similar. For all tests undertaken a small curvature is observed in the first part of the load vs axial displacement curves. This is due to small adjustments in the end-support devices that occur in the initial stage of the loading process. Hence, the actual axial displacement of the CFS column is slightly lower than the ones presented.

Observing the obtained results, it is clear the advantages of using built-up cross-sections in CFS building construction industry. For instance, for fixed-end support condition, the buckling load of columns with 2R cross-section

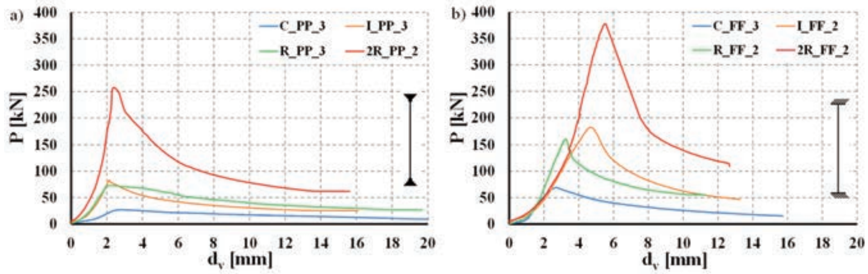

Figure 5. Load *vs* axial displacement diagrams for the compression tests undertaken on cold-formed steel columns with C, I, R and 2R cross-sections. a) Results for the pinned-end support condition; b) results for the fixed-end support condition.

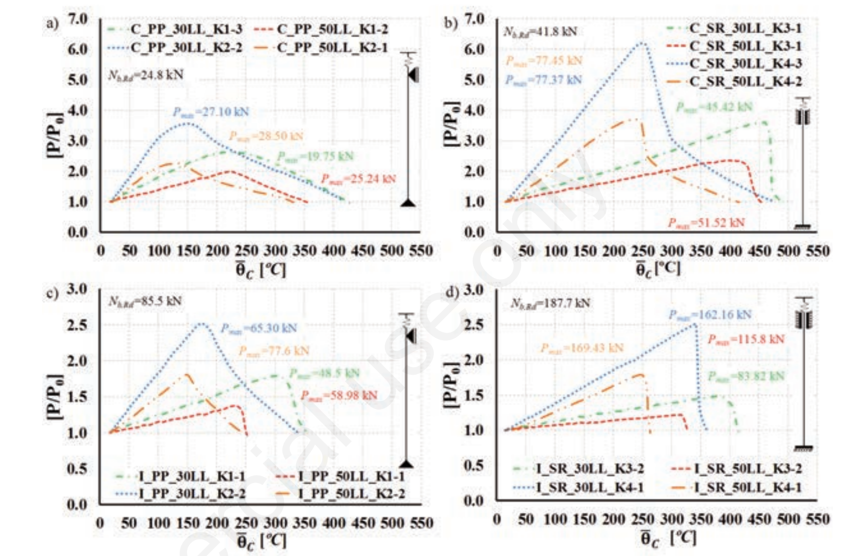

Figure 6. Non-dimensional restraining forces ratio for lipped and open built-up I columns. a) C, pinned-end support; b) C, semi-rigid end support; c) I, pinned-end support; d) I, semi-rigid end support.

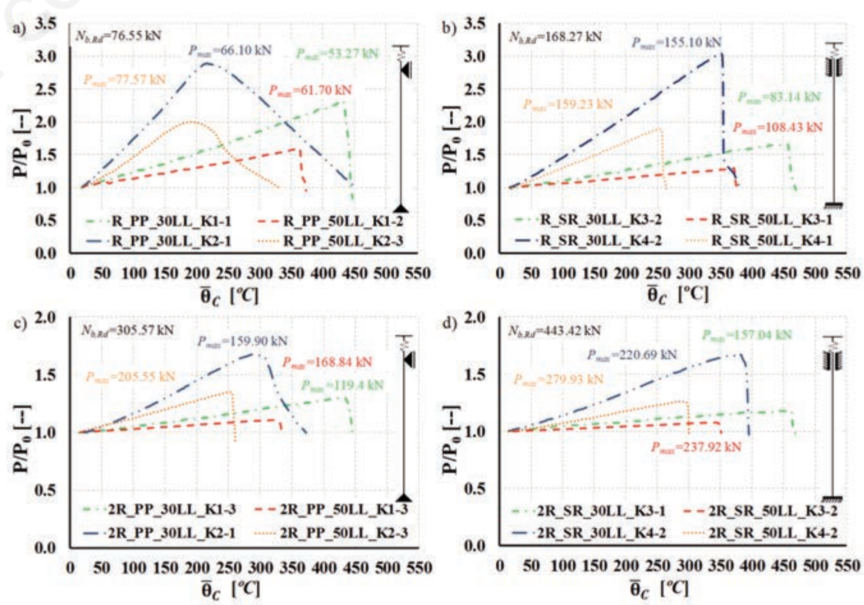

Figure 7. Non-dimensional restraining forces ratio for closed built-up cold-formed steel columns. a) R, pinned-end support; b) R, semi-rigid end support; c) 2R, pinned-end support; d) 2R, semi-rigid end support.

was 5.6 times higher than the buckling load of columns with lipped channels, 2.01 times higher than the buckling load of columns with open built-up I cross-section and 2.51 times higher than the buckling load of columns with closed built-up R cross-section, for the pinned-end support condition. Comparing both experimental results with the design buckling load ($N_{b,Rd}$) determined according the EN 1993-1-3:2006[9] some relevant differences were observed. For instance, it seems that the design-buckling load is too conservative for fixed-ended lipped channel columns. Also it was observed that increasing the number of profiles it seems that the design buckling predictions become unconservative, as it can be observed for columns with 2R cross-section.

Fire tests with restraint to thermal elongation

The evolution of restraining forces is a non-dimensional P/P_0 ratio presented as a function

of the mean temperature of the column ($\bar{\theta}_C$) for the different tested conditions. The mean temperature represents the integral of the weighted mean temperature calculated in each one of the instrumented sections along the length of the column. In these graphs it is possible to observe the expected behaviour of a column in a real structure as the effect of a surrounding structure was simulated with the 3D restraining frames. Each tested column was loaded with a compressive serviceability load (30 and 50% $N_{b,Rd}$) that was kept constant throughout the entire fire test. Due to the thermal action and since the column was axially restrained the axial compression force (restraining forces) on the column started to increase whereas the mechanical properties of CFS degraded with the temperature increase. After reaching a maximum (P_{max}) the restraining forces (P) started to decrease reaching the initial service load applied (P_0) to the CFS column. This point defines the critical time (t_{cr}) and temperature (θ_{cr}) as the failure criteria in these experimental tests. In Figures 6 and 7 some results concerning the evolution of the restraining forces are presented. Observing the obtained results the influence of the initial applied load is clear. Increasing the serviceability load from 30 to 50% $N_{b,Rd}$ critical temperatures and critical times decreased for all tested cross-sections. Analysing the results taking into consideration the imposed levels of restraint to thermal elongation to the CFS column it was observed that, generally, increasing the level of the imposed restraint to thermal elongation the critical times and temperatures decreased.

It was clearly observed for every tested conditions that increasing the level of axial restraint to thermal elongation from 3 to 13 kN/mm the generated restraining forces increase significantly and the maximum value was reached for much lower mean temperatures. For example, for the 2R cross-section and considering a 30% initial load level and semi-rigid end-support condition, increasing the level of restraint lead, on average, to a reduction of the peak temperature of 91ºC. Also, it was observed that the magnitude of the generated restraining forces was higher for the lower initial load level. For instance, the average magnitude of the generated restraining forces (P-P0) obtained for the R_SR_30LL_K3 tests was about 32.78 kN whereas for the R_SR_50LL_K3 tests was 25.33 kN.

In order to clarify the behavior of CFS closed built-up columns with restraint to thermal elongation in case of fire it is interesting to represent the observed critical temperatures (θ_{cr}) as a function of the ratio (α_k) between the axial stiffness of the surrounding structure to the CFS column ($K_{a,s}$) and the axial stiffness of the column ($K_{a,c}$) (eq. 1 and Figure 8). Considering the obtained results some general considerations may be presented. For isolated columns under fire conditions subjected to a low level of restraint to thermal elongation its failure is clearly controlled by temperature increase and by consequent degradation of mechanical properties of the S280GD+Z steel. The additional restraining forces are generated gradually during the heating phase. However, if an isolated column under fire conditions is subjected to high or very high levels of restraint to thermal elongation its failure may be controlled by the severity of the generated restraining forces during the heating phase. Higher levels of restraint will lead to higher rates of the generated restraining forces and as a consequence buckling load of the columns under these conditions may be reached for lower

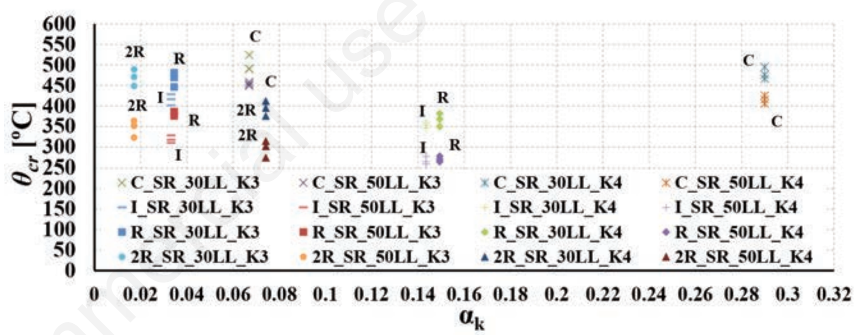

Figure 8. Variation of critical temperatures (θcr) as a function of the non-dimensional axial restraint ratio (αk) for columns with semi-rigid end-support condition.

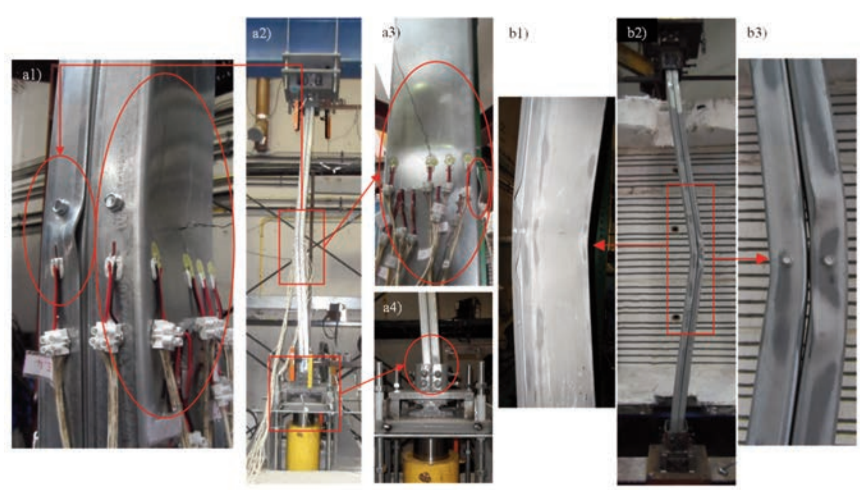

Figure 9. Failure modes observed for columns with closed built-up 2R cross-section and fix-ended support condition. a) Buckling tests at ambient temperature; b) fire tests with restraint to thermal elongation.

temperatures.

Failure modes

Generally, it was observed, for both ambient temperature tests and fire tests, that the failure modes were relatively similar for both test conditions. In Figure 9 some of the experimental failure modes observed for columns with built-up closed 2R cross-section and fixed-ended support conditions for both ambient temperature and fire tests. Regarding pin-ended columns, the predominant failure mode was flexural buckling about the minor axis, whereas for fix-ended columns the predominant failure mode was the interaction between flexural buckling about the minor axis and distortional and local buckling at about mid-height of the column and near the end-support devices.

Conclusions

This paper briefly reports a large experimental research on CFS columns at both ambient and fire conditions with restraint to thermal elongation. In buckling tests at ambient temperature it was clear the advantages of using built-up members was clear, since the increase in the buckling load was significant. The obtained results were then compared with the design buckling loads determined according to the EN 1993-1-3:2004.[9] For built-up columns generally it was found that design predictions are unsafe. It seems that by increasing the number of profiles the design predictions become more unsafe. This shows that additional research is needed in this field.

In the fire tests it was found that the interaction between the initial applied load and the imposed level of restraint to thermal elongation may significantly influence the behavior of isolated CFS columns under fire conditions. If a column could freely expand when subjected to fire, no additional forces would be generated. However, when some level of restraint exists additional forces are generated, which may lead to premature collapse and consequently to lower critical temperatures. It seems that increasing the level of thermal restraint the failure of the columns may be controlled by the generated axial restraining forces whereas for lower levels of thermal restraint the failure is controlled by temperature increase and consequent degradation of the mechanical properties of the S280GD+Z steel. Also it was found that the behavior of CFS columns in fire with axial restraint to thermal elongation may be described as a function of the non-dimensional axial restraint ratio (αk). Increasing αk will lead to a reduction in the critical temperature. However, to fully understand the behavior of isolated restrained columns in fire it is necessary to test different αk values, ranging from 0 (column can freely expand) to 1 (column fully restrained).

References

1. Young B, Rasmussen JR. Tests of fixed-ended plain channel columns. J Struct Engin 1998;124:131-9.
2. Kesti J, Davies JM. Local and distortional buckling of thin-walled short columns. Thin-Walled Struct 1999;34:115-34.
3. Young B, Chen J. Column tests of cold-formed steel non-symmetric lipped angle sections. J Construct Steel Res 2008;64:808-815.
4. Li Y, Yao X, Shen Z, Ma R. Load-carrying capacity estimation on cold-formed thin-walled steel columns with built-up box section. In: Proceedings of the 12th International Specialty Conference on Cold-Formed Steel Structures. St. Louis, MO, USA. 2010, pp. 89-103.
5. Georgieva I, Schueremans L, Vandewalle L, Pyl L. Design of built-up cold-formed steel columns according to the direct strength method. Proc Engin 2012;40:119-24.
6. Li Yuanqui, Li Yinglei, Wang S, Shen Z. Ultimate load-carrying capacity of cold-formed thin-walled columns with built-up box and I section under axial compression. Thin-Walled Struct 2014;79:202-17.
7. Wilson, R, Guzmán, A. Evaluation of the slenderness ratio in built-up cold-formed box sections. J Construct Steel Res 2011;67:929-35.
8. Schafer BW. Review: the direct strength method of cold-formed steel member design. J Construct Steel Res 64;2008:766-78.
9. European Commission, 2004. Eurocode 3: design of steel structures, Part 1-3: general rules, Supplementary rules for cold-formed members and sheeting, EN 1993-1-3:2004. European Committee for Standardization, Brussels, Belgium.
10. European Commission, 2005. Design of steel structures. General rules. Structural fire design, EN 1993-1.2:2005. European Committee for Standardization, Brussels, Belgium.
11. European Commission, 2005. Eurocode 3: design of steel structures. General rules and rules for buildings, EN 1993-1-1. European Committee for Standardization, Brussels, Belgium.
12. European Commission, 2006. Eurocode 3: design of steel structures, Part 1-5: plated structural elements, EN 1993-1-5. European Committee for Standardisation, Brussels, Belgium.