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Cold-formed steel columns made with open cross-sections subjected to fire

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ABSTRACT

An experimental study on the fire behaviour of cold-formed steel lipped channel (C) and built-up I (2C) slender columns with restrained thermal elongation is presented. The studied parameters were the stiffness of the surrounding structure, type of cross-section, end support conditions and initial applied load level on the columns. The results showed that increasing the stiffness of the surrounding structure and initial applied load level for the semi-rigid support conditions and both cross-sections, lead to a significant reduction of the critical temperature whereas for the pin-ended support conditions the reduction is supposed to be smaller.

1. Introduction

Cold-formed steel (CFS) is a material commonly used for multiple construction applications. Recently the demand for CFS structures has increased significantly, as it has been recognized to be used effectively as primary structural elements, especially for residential, commercial and industrial buildings. Its growing popularity in building construction is due to its advantages over other construction materials such as lightness and consequent ease of erection and installation, economy in transportation and handling. Some disadvantages are related to the use of thinner wall sections leading to design problems which may not be found in traditional structural design of hot-rolled steel elements. CFS sections may be subjected to various types of buckling, including local, distortional, global and its interactions. The research and product development carried out in the past led to the implementation of design specifications for CFS structures at ambient temperatures, such as the EN 1993-1-3:2005 [1]. However, so far, there is a lack of research on the structural behaviour of CFS structures under fire conditions. As a result, the design guidelines of such structural members at elevated temperatures are not accurate and precise enough to be used by designers. Currently the methods presented in EN 1993-1-2:2005 [2] for the hot-rolled steel members are also applicable to CFS members with class 4 cross-sections, establishing the same reduction factors for the yield strength of the steel and limiting the critical temperature to 350 °C. Some studies carried out in recent years have shown that the reduction of the mechanical properties of CFS as a function of the temperature has been different from those presented in EN 1993-1-2:2005 [2–5] and that limiting the temperatures to 350 °C without considering the load ratio may be overly conservative [6,7].

The great majority of experimental studies at elevated temperatures have been performed on CFS short columns in order to evaluate the local and distortional buckling phenomena individually and in combination [6–12] using experimental and numerical analysis [13–15]. Concerning built-up cross-sections commonly used in building construction the lack of research is more substantial even at ambient temperatures [16,17]. Also the influence of the surrounding structure on the structural behaviour of CFS columns has been neglected. Some studies regarding the influence of axial and rotational restraint have been conducted for hot rolled steel sections [18–28]. It is clear that if the column is thermal restrained the axial compression force (restraining forces) will increase up to a maximum in the column, depending on the stiffness of the surrounding structure and the initial applied load level, decreasing later due to degradation of the mechanical properties of the steel with temperature and this may lead to a premature collapse of the column in fire.

Feng et al. [8] performed load bearing capacity tests at ambient and elevated temperatures on compressed short CFS lipped channel members to study the local and distortional buckling behaviour. The experimental tests at elevated temperature were carried out under steady state conditions for different temperature levels without thermal restraint. This

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Nomenclature

CFS	cold-formed steel
CV	coefficient of variation
h	height of the cross section
I_c	moment of inertia of the column around the minor axis
$K_{a,s}$	axial stiffness of the surrounding structure
$K_{r,s}$	rotational stiffness of the surrounding structure
$K_{a,c}$	axial stiffness of the column
$K_{r,c}$	rotational stiffness of the column
L_c	column length
$N_{b,Rd}$	design buckling resistance of a compression member
N_{cr}	elastic critical force for the relevant buckling mode
$N_{c,Rd}$	design resistance of a cross-section for compression
P	axial restraining force generated in the column
P_{max}	maximum restraining force generated in the column
P_0	initial applied load on the column
RF.i	restraining frame number *i*
T	mean furnace temperature
T.i	thermocouple *i*
t_{cr}	critical time of the column
t_n	nominal thickness of the cross-section
t_{peak}	time for which the maximum restraining forces in the column is reached
α	non-dimensional axial restraint ratio
ρ_i	non-dimensional rotational restraint ratio in direction *i*
$\overline{\theta}_C$	mean temperature of the column
$\overline{\theta}_s$	mean temperature of the column cross-section
θ_{cr}	critical temperature of the column
θ_{cr}^{max}	maximum value of the critical temperature of the column
θ_{peak}	column temperature when the maximum restraining force is reached
θ_{peak}^{max}	maximum column temperature when the maximum restraining force is reached
$\overline{\lambda}$	non-dimensional slenderness
μ	mean value
σ	standard deviation

experimental study showed that depending on the initial imperfections the failure modes of nominally identical columns can be different at both ambient and elevated temperatures. Despite the difference in the failure modes the buckling loads of nominally identical columns were very close. This fact was also observed in the experimental tests conducted by Ranawaka and Mahendran [6] on CFS compression members under simulated fire conditions for studying the effect of the distortional buckling. Despite the three different types of distortional failure modes observed, the failure loads were about the same which means that the imperfections may influence the type of distortional buckling mode. It was stated that a limiting temperature method as the one proposed by the Steel Construction Institute (SCI) [29] for fire safety design of CFS members may be appropriate but more data should be needed to confirm it. This method indicates that the limit temperature (e.g. beams supporting concrete slabs – 550 °C; beams supporting timber floors – 500 °C; stocky columns – 500 °C; slender columns – 450 °C) must not be exceeded during the required period of fire resistance and depends on the relationship between the load supported on fire situation and the load-bearing capacity at room temperature. Comparing numerical with analytical results, including the effective width (EWM) [1] and the direct strength (DSM) methods [30,31], some studies [6,9,13] showed that these ones can be useful to predict the ultimate loads of short CFS columns at elevated temperatures.

The overview of the state of the art showed that the research on CFS slender columns under fire conditions is still scarce. For instance, commonly the predominant buckling modes have only been studied individually without considering their possible interaction and the influence of the surrounding structure on the structural behaviour of CFS columns in fire has not been yet studied.

Therefore, an experimental study on CFS columns in fire was performed at Coimbra University, Portugal [32,33]. Lipped channel (C) and open built-up I (2C) cross section columns with restrained thermal elongation were tested in fire situation. The tested parameters were the type of cross-section, stiffness of the surrounding structure to the column, load level and end-support conditions.

2. Experimental tests

2.1. Experimental set-up

The experimental set-up comprised a two dimensional (2D) reaction steel frame ((1) in Fig. 1(a) and (b)) and a three dimensional (3D) restraining steel frame adaptable for different levels of stiffness ((2) in Fig. 1(a) and (b)) in order to simulate the axial and rotational restraint of a surrounding structure to a CFS column in fire. The 2D reaction frame was composed by two HEB 500 columns 6.6 m tall and by one HEB 600 beam 4.5 m long using M27 grade 8.8 bolts in the connections. The levels of stiffness of the surrounding structure adopted in the experimental tests were chosen after conducting numerical simulations considering a residential CFS building with two storeys and three bays. The height of the columns was 3.0 m and the length of the beams was 5.0 m spaced from 0.6 m. Regarding the results obtained in these simulations it can be stated that 3 kN/mm of axial stiffness of the surrounding structure to the CFS column is a low value whereas 13 kN/mm is a medium/high value for the adopted experimental conditions. To achieve the desired levels of stiffness of the surrounding structure, in order to provide axial and rotational restraint (K_1–K_4) to the thermal elongation of the CFS columns, two different 3D restraining frames were used in the experimental tests. The first one (RF.1) was composed by four columns HEA 200 and four beams HEA 200 while the second one (RF.2) was composed by four columns HEB 300 and four beams HEB 300 (2 in the top and 2 in the bottom) arranged orthogonally. This restraining system intended to reproduce the actual boundary conditions of a CFS column when inserted in a real building structure. In order to confirm the levels of stiffness, beyond numerical simulations, some experimental tests at ambient temperature 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 displacement, on the point of intersection of the top beams of the 3D restraining frame, were 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