Thin-Walled Structures 74 (2014) 104–117

Contents lists available at ScienceDirect

Thin-Walled Structures

journal homepage: www.elsevier.com/locate/tws

Experimental analysis on cold-formed steel beams subjected to fire

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ARTICLE INFO

Article history:
Received 3 August 2013
Accepted 5 September 2013
Available online 19 October 2013

Keywords:
Cold-formed
Steel
Beam
Flexural
Fire
Buckling

ABSTRACT

The great majority of the studies in this area emphasise further the structural behaviour of cold-formed steel members by means of analytical approximation and purely numerical methods. In addition, they generally only take into account the structural behaviour of members with just one profile. On the contrary, this paper reports a series of flexural tests under fire conditions focused on cold-formed galvanised steel beams consisting on compound cold-formed steel profiles which are often used in floors and roofs of warehouses and industrial buildings. The main objective of this research was to assess the failure modes, the critical temperature and the critical time of the studied beams. Other important goals of this research work were also to investigate the influence of the cross-sections, the axial restraint to the thermal elongation of the beam and the rotational stiffness of the beam supports. Finally, the results showed above all that the critical temperature of a cold-formed steel beam might be strongly affected by the axial restraint to the thermal elongation of the beam.

1. Introduction

The use of cold-formed steel (CFS) profiles in buildings is a solution that is under continuous development, and so it will remain a challenge for the next few years. They are increasingly becoming a popular material in construction because they provide a high strength to weight ratio, they are easy to produce, transport and assemble when compared to heavier hot-rolled steel members. Studies in this area are still few, and mostly of a numerical nature. However, there are some that address the most important phenomena related to these elements at room temperature, including post-buckling resistance (local and distortional buckling) [1], global flexural, torsional and flexural-torsional buckling [2], shear resistance of stud walls [3], bending resistance [4] and resistance of some type of screwed connexions [5].

Cold-formed steel behave quite differently from hot-rolled steel members, since the latter are mostly found in class 1 or 2 cross-sections, while the former are class 3 or 4, according to EN1993-1.1 [6]. This is due to the high slenderness of the cross-section's walls (high ratio width/thickness of the wall) and the low torsional stiffness (much lower than the flexural stiffness), and to the fact that in many of these cross-sections, the shear centre does not coincide with the centre of gravity and the great majority of the cross-sections are open and either mono symmetric or completely asymmetric. Consequently, these members may buckle at a stress level lower than the yield point of steel. It is therefore clear that cold-formed steel members are more susceptible to instability (local and global) than hot-rolled ones, and there are still many open questions to investigate. As it is an emerging technology and since a great variety of profiles with different geometric shapes can be easily produced, it is of the utmost importance that studies in this field should be undertaken.

When it comes to fire, there are even fewer studies related to the behaviour of cold formed steel elements subject to high temperatures. Fire is another phenomenon which deteriorates its structural behaviour. The high thermal conductivity of steel and the high section factor of these structural members (very thin wall thickness) can lead to a rapid rise in steel temperature in a fire and together with the deterioration of its mechanical properties as a function of temperature may cause serious deformation of structural members and the premature failure of the building, like it happens with the hot-rolled steel structural members [7]. Since the structural behaviour of usual bare steel members under fire conditions may constitute a limiting ultimate limit state condition, investigations into CFS members under fire are required, especially because there are no simplified calculations methods for fire design of CFS structures, unlike for hot-rolled steel members, see EN1993-1.2 [8].

There are also only a few numerical studies on the behaviour of CFS members subjected to elevated temperatures. Some studies concluded that the increase of the magnitude of the local imperfections may lead to a relatively straightforward decrease of the initial stiffness of the member while the magnitude of the global imperfections may have more influence on the ultimate load of the member [9]. For example, Kaitila [9] observed that the ultimate compressive load of a C column may be reduced by 5.1% when the

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0263-8231/$ - see front matter
http://dx.doi.org/10.1016/j.tws.2013.09.006

Nomenclature

CFS	cold-formed steel
CV	coefficient of variation
L	beam span
$M_{b,Rd}$	design value of the resistant buckling moment
M_{cr}	critical elastic moment for lateral-torsional buckling
M_R	restraining moment at beam support
M_{Rd}	section moment capacity about the strong axis
N_A	axial restraining forces generated in the beam
P_0	initial applied load on a beam
T	mean furnace temperature
D_{S1}	vertical displacement of the beam at mid-span (section S1)
h	height of the cross-section
k_a	axial restraining to thermal elongation of the beam
$k_{a,b}$	axial stiffness of the beam
k_r	rotational stiffness of the beam supports
$k_{r,b}$	rotational stiffness of the beam
t_{cr}	critical time of the beam
t_n	nominal thickness of the cross-section
t_{N_max}	time when the maximum restraining force in the beam is reached
$\bar{\theta}_B$	mean outer beam temperature
θ_{cr}	critical temperature of the beam
θ_{N_max}	beam temperature when the maximum restraining force is reached
θ_S	steel temperature
$\bar{\theta}_S$	mean outer steel temperature
$\bar{\lambda}_{LT}$	non-dimensional slenderness for lateral-torsional buckling
μ	mean value
σ	standard deviation

global imperfection magnitude increases from $L/1000$ to $L/500$ and may also be reduced by 3.9% when the local imperfections magnitude increases from $h/200$ to $h/100$ at 600 °C. However, failure by distortional buckling may be further affected by the initial geometric imperfections. Ranawaka and Mahendran [10] noted that the maximum load capacity of a C column may be reduced by 20% and 30% when the distortional imperfections magnitude increases from zero to $2t_n$ at 20 °C and 500 °C, respectively. In addition, it seems that the design method given in EN1993-1.2 [8] is over-conservative for all temperatures except for CFS beams with very high slenderness values [11]. Kankanamge and Mahendran [11] also concluded from a parametric study that the EN1993-1.3 design methods [12] with buckling curve b are unsafe or over-conservative for some temperatures, especially in the intermediate slenderness region. Therefore, they proposed the use of other buckling curves for different temperature ranges for the fire design of CFS lipped channel beams. It is noticed that the methods established in Eurocode 3 were investigated by these authors in fire design by using the CFS mechanical properties at elevated temperatures [13]. With regard to the maximum temperature in CFS members, EN1993-1.2 [8] has enforced a limit of 350 °C, which seems to be overly conservative [14], especially, on the fire behaviour of beams. For example, it was found out by Laím and Rodrigues [15] and Lu et al. [16] that beams under certain boundary conditions retain load-carrying capacity until 700 °C.

Another important thing to point out is that most scientific investigations in this area are based on the structural behaviour of simple members (composed of just one CFS profile), in opposition to this study, where the fire performance of CFS beams consisting on compound cross-sections of profiles connected between them, which are often used in roofs and floors of industrial buildings, is presented. This paper therefore intends to fill the knowledge gap in this almost unexplored field and bring a better understanding about these issues. So, this paper is mainly aimed at the structural performance of cold-formed galvanised steel beams under fire conditions, based on the results of a large programme of experimental tests. The influence of the cross-sections, the axial restraint to the thermal elongation of the beam and the rotational stiffness of the beam supports was investigated. Furthermore, as reference, four-point bending tests at ambient temperature were also carried out for comparison, especially the failure modes of the corresponding beams [17]. Other important goal of this research is to provide experimental data for future numerical studies, in order to carry out a parametric study outside the bounds of the original experimental tests. Finally, the experimental and numerical results will be the basis of an analytical study for the development of simplified calculation methods for fire design of cold-formed steel beams.

2. Experimental tests

2.1. Test specimens

The specimens consisted of beams made of one or more CFS profiles, namely, channel and lipped channel profiles, also known as U and C profiles, respectively (Fig. 1). All these cross-sections were 2.5 mm thick and 43 mm wide. The insides bend radius and the length of the edge stiffeners of the C profiles were 2 and 15 mm, respectively. The C sections were 250 mm tall, whereas the U sections were 255 mm tall, so that the C profiles could be placed between the flanges of the U profiles (R beam), as illustrated in Fig. 1. Therefore, as it can also be seen in this figure, the compound lipped I beams consisted of two C connected in the web, whereas the compound R beams consisted of one C profile and one U profile connected in the flanges. The compound 2R beams were made of two R beams connected together by the C profiles' web. The total beam length was 3.6 m for all specimens, but the span was only 3 m in such a way that the beams and their supports could be accommodated by the horizontal electric fire resistance furnace available in the laboratory.

In addition, the profiles were screwed together as indicated in Fig. 1 by means of Hilti S-MD03Z 6.3 × 19 carbon steel self-drilling screws in S235 steel, at sections 0.05 m and 1.15 m away from the ends of the beams so that the spacing of the screws along the beam was about 1 m ($L/3$) (Fig. 2). All the profiles were made of

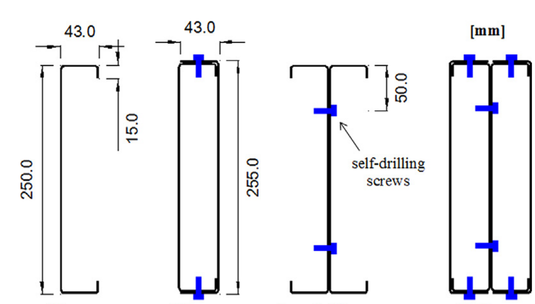

Fig. 1. Scheme of the cross-sections of the tested beams.