Journal of Building Engineering 8 (2016) 260–268

Contents lists available at ScienceDirect

Journal of Building Engineering

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

On the applicability and accuracy of fire design methods for open cold-formed steel beams

Luís Laím, João Paulo C. Rodrigues *

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

ARTICLE INFO

Article history:
Received 26 January 2016
Received in revised form
12 September 2016
Accepted 13 September 2016
Available online 14 September 2016

Keywords:
Fire
Cold-formed steel
Beam
Restrained thermal elongation
Design
Experimental
Numerical

ABSTRACT

This paper presents the results of an experimental and numerical investigation on the structural behaviour of cold-formed steel C and lipped-I beams subjected to uniform temperature distributions under standard fire conditions. A total of 18 specimens divided into four-point bending tests under fire conditions and under 3 different restraining conditions (including no restraints, partial axial restraint to the thermal elongation of the beam and both partial axial and rotational restraints at the beam supports) have been conducted. Local buckling, distortional buckling, lateral-torsional buckling and their interactions were observed in the tests. Then, the tests were modelled by the finite element programme Abaqus and, at the end, the numerical results showed good agreement with the experimental results in terms of axial restraining forces, vertical displacements, critical temperatures and buckling modes. The simulated results were still compared with the predictions from the currently European design rules (EN 1993-1.2:2005), in order to observe if there are safe and consistent regulations for fire design of these members. Finally, the numerical simulations have mainly shown that these design methods for CFS beams may be quite unsafe or over-conservative depending strongly on their boundary conditions.

1. Introduction

Cold-formed steel (CFS) members under high levels of compression may easily exhibit local (wall transverse bending only), distortional (both wall transverse bending and cross-section distortion) and global (lateral-torsional) buckling [1–4]. Local buckling is particularly prevalent and is characterised by the relatively short wavelength buckling of individual plate elements. Distortional buckling involves both translation and rotation at the compression flange/lip fold line of the member [5]. These special buckling modes are the most interesting and complex subjects within this research field. Beyond them, interactive buckling modes between or among the above ones are the most frequently in the CFS flexural members. This is why, the strength calculations of cold-formed steel members are carried out at several levels of complexity depending on the purpose of its use. For the standardised design of flexural members at ambient temperature the Effective Width Method (EWM) and the recently developed Direct Strength Method (DSM) [6] may be applied. The EWM is formally available in the EN 1993-1.3:2004 [7], EN 1993-1.5:2006 [8] and in the AISI S100-1996 [9], whereas the DSM is available in the Appendix 1 of the North American Specification [10] and the Australia/ New Zealand Standard [11]. The EWM, introduced by Von Kármán et al. in 1932 [12], performs a reduction of the plates that comprise a cross-section based on the stability of the individual plates for the prediction of the local buckling strength. It is noticed that this method is a semi-empirical calibrated formulation which takes into account the local buckling effects for thin-walled sections, but does not have sufficient procedures for predicting the distortional buckling failure. However, the EN 1993-1.3:2004 [7] provides specific provisions for the distortional buckling strength of CFS flexural members. This method adopted in the Eurocode considers the distortional buckling by using a reduced thickness in the calculation of the effective area of the edge stiffener and the distorted part of the compression flange. The reduction factor of thickness for distortional buckling depends among other parameters on the elastic buckling stress of the edge stiffener and the material yield strength. On the other hand, the DSM was initially proposed by Schafer and Peköz in 1998 [13] and it is based on the member elastic stability in contrast to the EWM. The essential difference between these two methods is therefore the replacement of plate stability by member stability. First, all elastic instability loads (or moments) for the gross cross-section should be determined (local, distortional and global buckling mode) as well as the load (or moment) that causes the section to yield. Then the member strength can be directly determined by predicting the load (or moment) capacities separately for global,

* Correspondence to: Departamento de Engenharia Civil, Faculdade de Ciências e Tecnologia da Universidade de Coimbra, Rua Luís Reis Santos, Pólo II da Universidade, 3030-788 Coimbra, Portugal.
E-mail address: jpaulocr@dec.uc.pt (J.P.C. Rodrigues).

http://dx.doi.org/10.1016/j.jobe.2016.09.004

Nomenclature

CFS cold-formed steel
DSM direct strength method
EWM effective width method
FEA finite element analysis
E modulus of elasticity of steel
L beam span
$M_{b,Rd}$ design value of the resistant buckling moment
$M_{b,\ fi,t,\ Rd}$ design lateral-torsional buckling resistance moment at time t of a laterally unrestrained beam in fire situation
$M_{fi,\theta,\ Rd}$ design moment resistance of the cross-section at temperature θ
M_{cr} elastic critical moment for lateral–torsional buckling
M_y appropriate bending moment
P_0 initial applied load on the beam
W_{eff} effective section modulus of the cross-section
W_y appropriate section modulus of the cross-section
f_y yield strength of steel
h height of the cross-section
$k_{E,\theta}$ reduction factor for the modulus of elasticity of steel at temperature θ
k_a axial restraint to the thermal elongation of the beam
$k_{a,b}$ axial stiffness of the beam
k_r rotational stiffness of the beam supports about the major axis
$k_{r,b}$ rotational stiffness of the beam about the major axis
$ky_{,\theta}$ reduction factor for the yield strength of steel at temperature θ
t thickness of the cross-section
$\gamma_{M,\ fi}$ partial material safety factor in fire design
θ_{cr} critical temperature of the beam
$\bar{\lambda}_{LT}$ non-dimensional slenderness for lateral-torsional buckling at ambient temperature
$\bar{\lambda}_{LT,\theta}$ non-dimensional slenderness for lateral-torsional buckling at temperature θ
$\chi_{LT,\ fi}$ reduction factor for lateral-torsional buckling in the fire design situation

local and distortional buckling, in the same way of EN 1993-1.1:2004 [14] predicts the design buckling resistance of a hot-rolled steel member, in other words, basing on reduction factors for the corresponding buckling curves and taking also into account the post-buckling reserve and the interaction between these modes.

When it comes to fire, the design rules are commonly based on past research on hot-rolled steel members. Note that due to the high slenderness of the cross-section's walls (high ratio width/thickness of the wall), CFS sections are mostly found in class 4 cross-sections, in contrast to hot-rolled steel sections which are mostly class 1 or 2. However, the simplified design methods presented in the EN 1993-1.2:2005 [15] can be used for CFS members according to its Annex E, but the area of the member cross-section must be replaced by the effective area and the section modulus by the effective section modulus, determined in accordance with EN 1993-1.3:2004 [7] and EN 1993-1.5:2006 [8], i.e. based on the material properties at 20 °C. Besides, the design yield strength of steel should be taken as the 0.2% proof strength. To further exacerbate the situation, EN 1993-1.2:2005 [15] still recommends a limit of 350 °C for the maximum temperature of members with class 4 cross-sections, which seems to be overly conservative [16–18]. Therefore, it is the objective of this research to investigate the accuracy of the current design guidelines for open CFS flexural members subjected to uniform temperature distributions under standard fire conditions. To accomplish this goal, a series of experimental tests and numerical simulations were performed at Coimbra University (UC) in Portugal on CFS beams with different open cross-sections (C and lipped-I sections) and boundary conditions (different levels of axial and rotational restraints at beam supports). These results were thereby compared with the predictions from the currently European design rules (EN 1993-1.2:2005 [15]), in order to observe if there are safe and consistent regulations for fire design of these members and for providing economical CFS structures in case of fire. Still note that, in the near future, these studies will be the basis of an analytical study for the development of simplified calculation methods for fire design of axially and rotationally restrained CFS beams.

2. Experimental and numerical models

2.1. Testing details

The experimental tests on CFS beams subjected to uniform temperature distributions under standard fire conditions were conducted at the Laboratory of Testing Materials and Structures (LEME) of Coimbra University (UC), in Portugal. The experimental programme consisted of 18 experimental tests on CFS beams, composed of just one (C beam) and two (2C beams) profiles. Six of which were just simply supported beams (Fig. 2a), 6 others were the same beams but with restrained thermal elongation (Fig. 2b), and the others were beams with axial and rotational restraint (Fig. 2c). All these profiles had the same nominal thickness (2.5 mm), nominal flange width (43 mm) and inside bend radius (2 mm). The edge stiffeners had 15 mm long and the nominal web

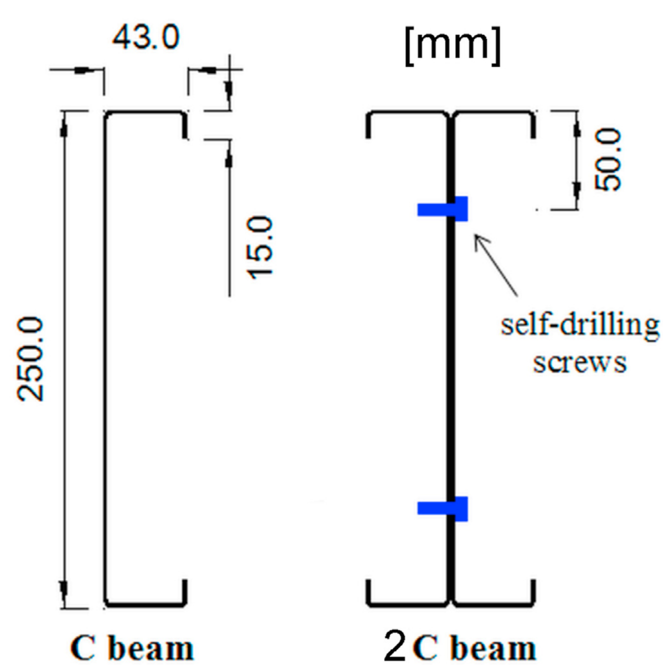

Fig. 1. Scheme of the cross-sections of the tested beams.