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Experimental and numerical analysis on the structural behaviour of cold-formed steel beams

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ABSTRACT

A research study on the structural behaviour of cold formed steel beams with C-, I-, R- and 2R-shaped cross-sections at ambient temperature is presented, based on the results of a large programme of experimental tests and numerical simulations. Firstly, several four-point bending tests were carried out in order to assess mainly the failure loads and failure modes of the beams. Secondly, a suitable finite element model was developed to compare with the experimental results, and finally, a parametric study was undertaken in order to investigate the influence of the thickness, height and length of the beams on its structural behaviour.

1. Introduction

Studies on the structural behaviour of cold-formed steel (CFS) beams are increasingly popular in the last decades. Instability phenomena, such as local, distortional, lateral-torsional buckling and their interactions, are the most interesting and complex subjects within this research field [1–4]. Understanding and dealing with these phenomena has been the central focus of recent research efforts.

These buckling modes are mostly responsible for the ultimate strength of the compression members as they may occur even before parts of the cross-section yield. The low torsional stiffness, the high slenderness and the geometric imperfections that are characteristic of CFS members are some of the main causes for their high susceptibility to buckling [5]. It is further noticed that the cross-section's walls have a high width-to-thickness ratio (the thickness is usually less than 3 mm), that the most cross-sections are open and/or asymmetric; in other words, there is no coincidence between the shear centre and the centroid of the section. Finally, the amplitudes of the geometric imperfections are normally of the same order of magnitude or higher than the thickness of the steel plates. Therefore, cold-formed steel members are usually classified as class 3 or 4 cross-sections, according to EN1993-1.1 [6].

The majority of studies in this field emphasise further the structural behaviour of these members by means of analytical approximations and purely numerical methods. The effective width method (EWM), which was included in the EN1993-1.3 [7], is an example of an analytical approximation method, whereas the commercial software ABAQUS [8] and the free software CUFSM [9,10] are examples of numerical methods. As numerical techniques for finding approximate solutions to partial differential equations and their systems, as well as integral equations, the programs ABAQUS and CUFSM use respectively the finite element method (FEM) [11] and the finite strip method (FSM) [12,13].

Furthermore, the direct strength method (DSM) [14], which was included in Appendix 1 of the North American Specification for the design of cold-formed steel structural members (AISI S100-2007) [15], and the effective section method (ESM) [16] are examples of combining both analytical approximations and numerical methods in order to assess the axial compressive and the flexural strength of the CFS members. It is noticed that the EWM and the DSM are the design methods commonly used by designers, although their application is not easy. The EWM performs a reduction of the plates that constitutes a cross-section based on the stability of the individual plates for the prediction of the local buckling strength. In addition, this method considers distortional buckling by using a reduced thickness in the calculation of the effective area of the edge stiffener under compression. The thickness reduction factor depends, among other parameters, on the elastic buckling stress of the edge stiffener and the material yield strength. This distortional buckling strength is obtained by an iterative procedure, especially for flexural members since the

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Nomenclature

C_b	buckling factor
d_{S1}	vertical displacement of the specimen at mid-span (section S1)
E	Young's modulus
f_y	yield strength of steel
G	shear modulus
h	height of the beam
I_y	moment of inertia about the minor axis
I_w	warping section constant
J	torsion section constant
L	length of the beam
k_y	effective length factor
k_w	warping effective length factor
M	bending moment
$M_{b,RD}$	design value of the resistant buckling moment
M_{cr}	critical elastic moment for lateral-torsional buckling
M_{Rd}	resistant moment of the gross or effective cross-section
$M_{xx'}$ – S.M.	measured bending moment about the xx' axis
$M_{yy'}$ – S.M.	measured bending moment about the yy' axis
$M_{xx'}$ – T.C.M.	calculated bending moment about the xx' axis
$M_{yy'}$ – T.C.M.	calculated bending moment about the yy' axis
P	applied load on the beams
P_{max}	maximum load-carrying capacity of the beam
PW	maximum load-carrying capacity of the beam including its self-weight
t	nominal thickness of the beam cross-section
SW	self-weight of the beam
W_{eff}	effective section modulus of the beam
W_{el}	elastic section modulus of the beam
W_y	section modulus of the beam
α	imperfection factor
β_{S1}	lateral rotation of the beam at mid-span (section S1)
χ_{LT}	reduction factor for lateral-torsional buckling
ε_{S1}	measured strain in the beam at mid-span (section S1)
γ_{M1}	partial factor for resistance of members to instability
$\bar{\lambda}_{LT}$	non-dimensional slenderness for lateral-torsional buckling
θ	rotation of the beam supports

stress distribution over the new effective cross-section may be different from the effective cross-section previously calculated. On the other hand, a linear elastic stability analysis is the main idea behind the DSM. First, all elastic instability modes for the gross cross-section are determined (local, distortional and global buckling mode). Subsequently, based on reduction factors for the corresponding buckling curves, the predictable strength of the CFS members is calculated. Here, the designers have to get computational tools for the calculation of the elastic buckling loads of the members, which can be determined either by FEM software such as ABAQUS or, in a speedier way, using CUFSM. The essential difference between these two methods is the replacement of plate stability with member stability.

Most of the studies in the literature only take into account the structural behaviour of CFS members with just one profile and the majority of them are of numerical nature [17–19]. The present paper reports on a series of flexural tests at ambient temperature focused on cold-formed steel beams consisting of compound cross-section CFS profiles which are often used in roofs of industrial buildings. The study involved both experimental and numerical investigations. The main objective of this research was to compare the structural response of the different kinds of beams and also to compare the results with the predictions from available design rules.

To conclude, in the near future, this work will be followed by an experimental and numerical study on the behaviour of such beams under fire conditions with the purpose of developing simplified calculation methods for fire design of cold-formed steel beams since there is nothing related to the fire design of these elements in EN1993, parts 1.2 [20] and 1.3 [7].

2. Experimental tests

The experimental tests on cold-formed steel beams were conducted in the Laboratory of Testing Materials and Structures of the University of Coimbra (UC), in Portugal. The experimental programme consisted of 12 quasi-static bending tests at ambient temperature, allowing the study of the flexural behaviour of 4 types of beams with different cross-sections. For each type 3 tests were carried out (B-C_*i*, B-I_*i*, B-R_*i* and B-2R_*i*, where *i* stands for the test number, $i=1$–3), in order to obtain a better correlation of the results.

2.1. Test specimens

The specimens consisted of beams made of one or more cold-formed steel profiles, namely, C (lipped channel) and U (channel) profiles (Fig. 1). 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 of the C profiles were 15 mm long and the nominal web depth was 250 mm for the C profiles and 255 mm for the U profiles. As it can also be seen in Fig. 1, 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 furnace available in the laboratory. As already stated, the following studies will address the fire behaviour of this kind of beams.

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

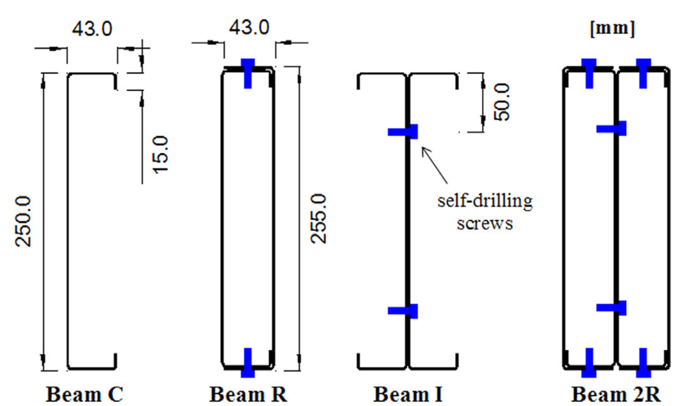

Fig. 1. Scheme of the cross-sections of the tested beams.