Numerical analysis of cold-formed steel sigma-shaped beams in fire conditions

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Abstract

This paper presents the development of a finite element model for predicting the flexural behavior of cold-formed steel beams subjected to fire and with restrained thermal elongation (different grades of axial and rotational restraint). The numerical simulations were carried out with the finite element software Abaqus/CAE. The numerical model was compared with experimental results in order to validate it for further parametric studies. This paper provides details of the simulation methodology for achieving the numerical stability and faithful representation of detailed structural behavior. To verify the accuracy of the Abaqus model the numerical and experimental results were compared in terms of axial restraining forces, vertical mid-span deflections, critical temperatures and failure modes of the beams. The numerical results showed a good agreement with the experimental ones. However, the critical temperatures determined numerically were slightly lower than the numerical ones in the simply supported and axially restrained beams and the opposite in the axially and rotationally restrained beams, this for both sigma and 2-sigma section beams.

Keywords Cold-formed steel · Beam · Fire resistance · Flexural behavior · Numerical analysis

Introduction

The use of cold-formed steel (CFS) members has increased significantly in the past few years due to the large number of advantages of this type of profiles when compared with other construction solutions. The great variety of profiles available allows the building of different member cross-sections and their high strength to weight ratio provides an easier production, assembly and transportation, making this material even more popular in construction. Nevertheless, as cold-formed steel members have high slenderness (width-to-thickness ratio) and low torsional stiffness, those members are very susceptible to instability phenomena and can fail in a variety of buckling modes including local, distortional and

global and their interactions, resulting in complex methods to calculate the strength of CFS members [1]. These buckling modes are mainly responsible for the ultimate strength of the compression members, once they may precede the cross-section yield.

So far, despite the growing popularity of these steel structures, the research on cold-formed steel structural elements under fire conditions is still scarce. Consequently, specific standards for fire design of CFS structures have not yet been established resulting in inadequate fire design guidelines for this type of members [2–5]. Therefore, it is extremely important to investigate and assess the structural performance of CFS flexural members under fire conditions.

In this context, the numerical modelling becomes hugely important, since a numerical model fulfil adequately the features of interest concerning the behavior of cold-formed steel members at high temperatures [6, 7]. The majority of the studies in this field only take into account the structural behavior of CFS beams at high temperatures with just one profile and the majority of them investigate unrestrained members [2, 8, 9], whereas just a few studies have been performed on restrained members, in which the effect of surrounding structures is taken into account [3, 5–7].

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The suitability of the current design guidelines for lateral-torsional buckling of CFS lipped channel beams under uniform bending moment and subjected to uniform high temperatures have already been investigated numerically [8]. In these researches, nonlinear finite element (FE) analyses of CSF beams under lateral-torsional buckling were undertaken using Abaqus software [10] and reduced mechanical properties were calculated by using the equations reported in previous works [11]. All the beams were considered with idealized simply supported boundary conditions and were loaded at two-quarter points of the beam. Initial imperfections and flexural residual stresses were considered while the membrane residual stresses were not considered since they decrease rapidly with increasing temperatures. They were also analyzed the effect of various parameters such as thickness, grade of steel and span of the beam on the structural behavior of CFS beams under fire conditions (with temperatures varying from 20 to 700 °C) under steady-state method. It was found that the limit of 350 °C for the maximum temperature of CFS members, recommended in EN1993-1-2 (2005) [12], seems to be overly conservative.

Besides this critical temperature suggested, the Annex E of EN1993-1-2 (2015) [12] establishes a design method that can be adapted to CFS members at high temperatures, since applied a reduction factor for the Young's modulus and yield strength of steel and the area of the member cross section and section modulus are replaced respectively by the effective area and effective section modulus, as established in EN 1993-1-3 (2004) [13] and EN 1993-1-5 (2006) [14].

The effect of the surrounding structure in the behavior of the CFS beams at high temperatures, under transient state method, have been also experimentally and numerically studied some years ago [15]. The effect of the axial and rotational stiffness on the behavior of the beam at high temperatures has been studied in what concerns to the failure modes, temperatures and critical times of CSF beams with different sections geometry (C, I, R and 2R) [5–7]. The studies have been also done for Σ -shaped beams under fire conditions. The critical time observed was less than 30 min for all the cross-sections and the critical temperature of all CFS beams was up to 700 °C, which was much higher than recommended by EN1993-1-2 (2004) [12]. Based on the experimental results, FE analysis was performed to study the effect of the initial applied load level, length of the beam, level of axial and rotational restraints and section geometry (C, I, R and 2R) on the structural behavior of CFS beams under fire conditions [7]. In order to model the material non-linearity, Von Mises criteria and isotropic strain hardening were adopted. In accordance with numerical simulation results, the fire performance of hollow sections (R and 2R beams) showed an increase performance of up to 50% in comparison with open sections (C and I beams). Moreover,

it was possible to observe that the lateral-torsional buckling was the main failure mode responsible for the collapse of open cross-sections while closed profiles collapse due to distortional buckling.

By analyzing the studies conducted, they have mainly shown that the design methods proposed by EN1993-1.2 for CFS beams under fire conditions may provide quite unsafe or over-conservative results depending on relative slenderness, serviceability load and boundary conditions of the beams [15]. The analyzes have shown that the stiffness of the surrounding structures greatly affect the critical temperature of the beams. Finally, the studies suggested appropriate recommendations for the European fire design rules for beams with different cross-sections (C, I, R and 2R). However, the behavior of Σ and 2Σ -shaped beams need further investigations as the available data is scarce.

In this sense, the purpose of this paper is to describe the development of a finite element model suitable for predicting the flexural behavior of CFS Σ -shaped beams under fire conditions, as well as to investigate the influence of the axial and rotational restraining on the fire behavior of these beams. The numerical simulations were carried out with the finite element software Abaqus [10] and the numerical model was compared with some experimental results [15, 16], in order to validate the developed numerical model and consequently obtain reliable numerical results. The assessment of the accuracy of the developed numerical models was conducted by comparing the axial restraining forces, vertical displacements, critical temperatures and failure modes obtained from finite element simulations with the corresponding experimental test.

The present research, intended therefore to develop a calibrated numerical model that will be used to accurately conduct future numerical parametric studies to provide valuable data to help developing detailed understanding of the flexural behavior of CFS sigma-shaped beams subjected to fire [17].

Finite element modeling

In order to simulate the structural behavior of cold-formed steel beams under combined flexural loading and fire conditions, numerical models were developed using the finite element software Abaqus [10]—a powerful computational tool for modelling structures with material and geometric nonlinear behavior. The models were built based on experimental results previously carried out at University of Coimbra, Portugal [15, 16] and the calibration was done by comparing the numerical with the experimental results, in order to validate the developed numerical model and consequently obtain reliable numerical results. In this

section, a brief description of the experimental test set-up and procedure is presented followed by a detailed description of the modelling approach and its validation with the experimental results.

Experimental tests

The experimental tests have studied the behavior of the CFS sigma-shaped beams at high temperatures (Fig. 1) [15, 16]. The beams were composed of one or two sigma-shaped profiles (referenced as sigma beams and 2-sigma beams) with 255 mm tall, 70 mm wide, a nominal thickness of 2.5 mm and edge stiffeners with 25 mm tall. The span of the studied beams was 3000 mm and the built-up beams were connected by self-drilling carbon steel screws Hilti S-6.3 × 19MD03Z along the length of the beam spaced 650 mm apart. These sigma profiles were manufactured using high strength steel S320GD + Z275 (zinc coating thickness of 0.04 mm (275 g/m²), with a yield strength of 280 MPa and an ultimate tensile strength of 360 MPa [13].

The experimental study consisted on eighteen experimental tests. Six of which were just simply supported beams, six others were the same beams but with restrained thermal elongation, and the others were beams with axial and rotational restraint. The experimental program carried out, is defined in Table 1, where 2Σ_{ra+rr_1} refers the first test (1) of the 2Σ (2-sigma beam) with axial (ra) and rotational (rr) restraint.

The beams were supported by a roller and pinned support in order to provide a simply supported beam. Four restraining steel beams were provided at each end of the

Table 1 Test Plan (i = 1, 2 or 3)

Test reference	$\bar{\lambda}_{LT}$	$M_{b,Rd}$ (kN.m)	P_0 (kN)	ra (kN/mm)	rr (kN.m/rad)
Σ _i	1.08	11.24	11.24	0	0
2Σ _i	0.70	34.15	34.15	0	0
Σ _{ra_i}	1.08	11.24	11.24	15	0
2Σ _{ra_i}	0.70	34.15	34.15	15	0
Σ _{ra+rr_i}	1.08	11.24	11.24	15	150
2Σ _{ra+rr_i}	0.70	34.15	34.15	15	150

beam to simulate the axial restraint to the thermal elongation of the beam and the rotational stiffness of their supports, as schematically shown in Fig. 2. These experimental tests were performed in two stages: loading and heating stage. First, the beams were loaded at two points 1000 mm (one-third of the beam span) from the supports of the beam (four-point bending tests) until achieved 50% of the design value of the buckling load of the beams at ambient temperature (P_0) that was calculated according to the methods proposed in EN1993-1.1 (2005) [18]. The load was applied under load control at a rate of 0.1 kN/s. Finally, after the desired load level was achieved the specimens were heated up with an electrical furnace programmed to reproduce the standard fire curve ISO 834–1 (1999) [19]. During the heating period the load was kept constant until the specimen failure.

The tested beams were equipped with LVDTs in order to measure the axial and vertical displacements at mid-span and 500 mm away from the beam supports. Thermocouples were used to monitor the increase of temperatures at several points of the beams and inside the furnace. The load applied on the beams, the axial restraining forces and the restraining moments at beam supports were also measured during the tests.

The retrieved data from the experimental tests, were considered and used in the developing and calibration process of the numerical models presented in this article. Next, it is described in detail the parameters, considerations and assumptions took into account in the developed nonlinear finite element model to predict the fire performance of such beams.

Numerical modeling

Geometrical parameters

For the purpose of reproducing the behavior of the tested beams with great accuracy, a three-dimensional numerical model was created for replicating as close as possible the experimental test set-up described in Sect. 2.1 (Fig. 2). The CFS beams were created by defining the

Fig. 1 Dimensions of the cross-sections studied (a) single and (b) built-up

Fig. 2 Axial and rotational restraining system: 1: tested beam; 2: roller support; 3: pinned support; 4 and 5: axial restraining; 6 and 7: rotational restraining; 8, 9, 10 and 13—load cell; 11—concrete slab; 12 horizontal furnace; 14 hydraulic jack; 15 reaction frame (Laim et. al. 2015)

cross-sectional profile using center line nominal dimensions and extruding the cross-section in the longitudinal direction to the required length. Additionally, four rigid plates were initially generated separately and subsequently were attached to the beams to simulate the four-point bending test. Two of them were used to represent the beam supports and the other two were used to obtain a uniform distribution of the load across the surface of the beam. The 2-sigma section was back-to-back connected by screws which were created and assembled by translating and rotating them to the required location. The 3D model is shown in Fig. 3 [17].

In the numerical simulations have been used the nominal dimensions of the specimens that are similar to the experimental measured ones.

Loading, contact and boundary conditions

The applied load was simulated with concentrated nodal forces (Y-direction) acting vertically at the middle of rigid plates connected to the upper flange surface of the beams. The application points of the load were located at a distance of one-third of the beam span ($L/3$).

To simulate the required simply supported boundary conditions, a reference node in the middle of the rigid plate was selected to control the motion of that surface. The reference node used to represent the pinned support was constrained against all degrees of freedom excluding the rotational about X-axis. On the other hand, to simulate the roller support the displacement along horizontal axis (Z-direction) was not restrained too. In addition, to avoid the lateral deformation

Fig. 3 FE model developed for 2 sigma beams under fire conditions with axial and rotational restraint

of the beams, all nodes located at the edges of the supports were restrained against lateral translation (X- direction). Finally, the end sections of the finite element models were coupled to two concentric reference points, where a linear spring model was associated in order to achieve the same axial and rotational restraint conditions used in the experimental tests. The values of the stiffness of these axial springs were the same considered in the experimental tests.

Concerning the built-up sigma cross-section, following the methodology adopted in similar studies on CFS bolted profiles [5, 20, 21], the contact interactions between the web surfaces of the profiles (Fig. 4b) were defined by using the “surface-to-surface” contact discretization method with a finite-sliding tracking formulation, which is suitable for models with large plastic deformation. Concerning the contact interaction property, it was adopted a tangential friction coefficient of 0.2 for the behavior in tangential direction, “hard contact” properties were enforced for the normal direction behavior to minimize penetration of slave surfaces into master surfaces which might occur if plates bend significantly and the penalty method was defined as the constraint enforcement between the steel profile surfaces. A linear damping coefficient was also adopted. Additionally, to model the contact between the two CFS profiles and the self-drilling screws (Fig. 4c) a rough and hard contact were assumed, preventing the sliding between surfaces and allowing the transmission of compressive forces, respectively.

The temperature during the experimental tests were measured at different points of the cross-section of the specimen. So, in order to reproduce the non-uniform temperature distribution observed in the experimental tests, influence areas were defined depending on the position of the specific thermocouple. To simulate the heating of the beams, it was assumed that the temperatures in these areas were uniform and equal to the respective thermocouples and each measured temperature was inputted along that area as a function of time. It is important to mention that the temperature was considered uniform in the longitudinal direction of the beam and that the temperature was considered uniform and equal to 20 °C at beam supports that were thermally insulated in the experimental tests (Fig. 4a).

Finite elements and meshing

The cold-formed steel beams were modelled by using the general-purpose element S4R (4-node double curved general-purpose shell, reduced integration with hourglass control), which has been extensively and successfully used to model thin-walled structural elements under fire conditions [2, 5, 6]. This element type takes transverse shear deformation into account as well as the thick shell elements. Regarding the self-drilling screws used in built-up 2-sigma beams, the finite element chosen was the C3D8R (8-node hexahedral element, continuum, linear reduced integration with hourglass control).

Simulations have been conducted with the aim of defining an appropriate and accurate finite element model with a reduced computational time. Four mesh densities for the sigma sections at ambient temperature were examined (Table 2) and compared with experimental results conducted by Ferrer (2014) [16].

Comparing the loadbearing capacity of the beams and the deformed configuration for the four mesh densities it was found that good simulation results could be obtained by using finite element meshes of 10 × 10 mm or 10 × 30 mm. To save computational time, finite element mesh size of 10 mm × 30 mm for the flat plate sections was found to be optimal. However, to achieve better results, smaller elements were used to model the rounded corner regions (Fig. 5a and b). Regarding the screws, a mesh size of approximately 2 mm was assumed (Fig. 5c).

Figure 6 illustrates the numerical failure modes of the tested specimens under bending and a mesh size of

Table 2 Mesh Sensitivity

Size Mesh	Loadbearing capacity of the beams		Relative error [%]
	Numerical Results	Experimental results	
10 × 10 mm	34.68	34.37	1%
20 × 20 mm	32.67		-5%
30 × 30 mm	32.96		-4%
10 × 30 mm	33.89		-1%

Fig. 4 a Temperature non-uniform distribution introduced in FE model at 483 s of the analysis; and detail of the contact between b profiles and c self-drilling screws and the CFS profile

10 × 30 mm and they can be compared to the experimental results. The figure also presents the finite element results in the form of von Mises or minimum principal stresses to help understanding the behavior of the beams.

Material modelling

For elastic analysis the sigma beam was assumed to have linear elastic material with Young's modulus of 210 GPa and Poisson's ratio of 0.3. On the other hand, for non-linear

analysis, the material of the beam was modelled with von Mises criteria and strain hardening. The stress–strain curves of CFS material (Fig. 7a) were obtained from the model proposed by Laím et al. (2016) [7] and based on the real curves taken from tensile coupon tests [20]. This model is comprised of a gradual yielding behavior followed by a considerable period of strain hardening, with a nominal yield strength of 320 MPa, a tensile strength of 390 MPa and a Young's modulus of 210 GPa. For the steel screws it was adopted an elastic-perfectly plastic behavior [5]. Note that

Fig. 5 Finite element mesh of the sigma beams (a); 2-sigma beams (b) and the screw (c)

Fig. 6 Numerical (a) and experimental (b) results

Fig. 7 Mechanical and thermal properties: (a) Stress–strain curves; (b) Relative thermal elongation in function of temperature

Fig. 8 Evolution of the vertical displacements (**a1**, **a2**) and evolution of the axial restraining forces as a function of the mean temperature of the beams (**b1**, **b2**, **c1**, **c2**)

the residual stresses were not considered in this study as suggested by other researchers [5, 21, 22].

Engineering stress (also known as nominal stress) is the applied load divided by the original cross-sectional area of a material. True stress is the applied load divided by the actual cross-sectional area (the changing area with respect to time) of the specimen at that load. Since the analysis of post-buckling may involve large in-elastic strains, the nominal

(engineering) static stress–strain curve were converted to a true stress and logarithmic plastic strain curve. Figure 7a shows the stress–strain curves for different temperatures used as input in the numerical simulations.

The thermal elongation (Fig. 7b) was taken from the predictions proposed by Chen and Young (2007) [23], because the one established in Eurocode was found to be over-conservative [20]. The yield strength and Young's modulus

Table 3 Critical temperatures for sigma beams—Comparison between experimental and numerical results

Experimental tests	Critical temperature [°C]		Relative error [%]
	Experimen- tal results	Numerical results	
Σ_{01}	669	643	-3.89
Σ_{02}	680		-5.44
Σ_{03}	693		-7.22
Mean	681	643	-5.53
Σ_{ra_01}	526	531	0.95
Σ_{ra_02}	607		-12.52
Σ_{ra_03}	564		-5.85
Mean	566	531	-6.13
Σ_{ra+rr_01}	666	681	2.25
Σ_{ra+rr_02}	622		9.49
Σ_{ra+rr_03}	668		1.95
Mean	652	681	4.45

Table 4 Critical temperatures for 2-sigma beams—comparison between experimental and numerical results

Experimental tests	Critical temperature [°C]		Relative error [%]
	Experimental results	Numerical results	
2 Σ_{01}	594	585	-1.52
2 Σ_{02}	601		-2.66
2 Σ_{03}	632		-7.44
Mean	609	585	-3.94
2 Σ_{ra_01}	540	506	-6.30
2 Σ_{ra_02}	524		-3.44
2 Σ_{ra_03}	509		-0.59
Mean	524	506	-3.50
2 Σ_{ra+rr_01}	574	606	5.57
2 Σ_{ra+rr_02}	614		-1.30
2 Σ_{ra+rr_03}	626		-3.19
Mean	605	606	0.22

of the steel decrease with increasing temperature, thereby, appropriately reduced mechanical properties of steel with the high temperature, obtained from the annex E of EN 1993–1-2 (2005) [12], were introduced in the models.

Analysis Method

Finite element (FE) three-dimensional models were created for all tested cross-sections under fire conditions with different restraining conditions, including (i) no thermal restraint; (ii) partial axial restraint; and (iii) partial rotational restraint at the beam supports. It has been conducted two different types of numerical analyses, namely linear eigenvalue

buckling analysis and nonlinear buckling analysis. Firstly, a linear perturbation analysis was performed in order to establish the buckling modes and eigenvalues of an ideal linear elastic beam. Several possible buckling modes were obtained from the perturbation by using Eigen-modes prediction. From this step, it was chosen buckling mode shapes to introduce as initial geometric imperfections in the nonlinear analysis of the finite element model. The imperfection was applied using *IMPERFECTION keyword directly in the input file.

Some magnitude of the imperfections was introduced as parameter to calibrate the models, based on currently magnitudes applied in this type of members [2, 15, 18, 24]. After understanding their effects on structural response of this kind of beams and trying to entirely reproduce the final deformed shape observed in the experimental tests [15, 16] an interaction between different buckling modes were considered and the maximum values assumed for the magnitude of the imperfections: $L/500$ for global imperfections, $2t$ for distortional imperfections and $h/100$ for local imperfections, in which L , t and h represents the length, the thickness and the height of the beam's cross-section, respectively.

Upon incorporation of the geometric imperfections into the developed numerical models, nonlinear buckling analysis, accounting for both material and geometric nonlinearities, was carried out to replicate the full range of the fire-resistant tests for all the conditions tested in the experimental analysis. As in the experimental tests, this analysis occurs in two steps: first the serviceability load (P_0) was gradually applied on the beams and then it was conducted the heating phase where the load was kept constant until the structure becomes unstable, where the beam deformation was large enough, $L^2/(400 h) = 88$ mm (failure criteria in terms of deformation for flexural elements according to the ISO 834–1 (1999) [19]. The time instant when the beams reach this deformation, corresponding to the loss of their load-bearing capacity (failure criteria in terms of strength), was assumed as the critical time. Therefore, for this study it was assumed that the critical temperature corresponded exactly to the one measured at the critical time. A “static general” option featuring a full Newton–Raphson solution technique was used for the static nonlinear analysis of the beams due to this method having provision for numerical stabilization which were required during the analysis.

Validation of numerical models

Comparison between the experimental and finite element results

The assessment of the accuracy of the developed numerical models was conducted by comparing the

Fig. 9 Comparison between experimental and numerical failure modes for simply supported beams (a) sigma and (b) 2-sigma beams

Fig. 10 Comparison between experimental and numerical failure modes for axially restrained beams (a) sigma and (b) 2-sigma beams

Fig. 11 Comparison between experimental and numerical failure modes for axially and rotationally restrained beams (a) sigma and (b) 2-sigma beams

displacement-temperature curves of the simply supported beams (Fig. 8a1, a2), axial restraining force-temperature curves of the axially and rotationally restrained beams (Fig. 8b1, b2, c1, c2), critical temperatures (Tables 3 and

4) and failure modes (Fig. 8) obtained from finite element simulations with the corresponding experimental test results.

As observed in Fig. 8, the result of Abaqus simulation was relatively close to the experimental ones, since all

developed FE predicted curves were comparable to the experimental ones, especially in which concerns the maximum axial restraining forces supported by the beams and critical temperatures. As specified in Table 2, the mean differences between the critical temperatures obtained from the tests and FE analysis are less than 7%, indicating that the estimated results were in good agreement with the experimental ones. So, it is possible to state that the finite element developed can accurately reproduce the behavior of flexural CFS beams under fire conditions with axial and rotational restraint.

Comparison between the experimental and numerical failure modes

On the other hand, Figs. 9, 10 and 11 present the experimental deformed configuration of the tested beams and the corresponding deformation pattern predicted by the numerical analysis after the end of the test and analysis respectively, indicating an excellent agreement.

Overall, the FE models developed were capable of accurately replicating the experimental response of CFS beams under fire conditions, ensuring a strong validity of the developed FE models. As observed in the experimental tests and validated by the numerical analysis, the lateral-torsional buckling was the main failure mode responsible for the collapse of the beams. Besides the lateral-torsional buckling, failure modes of these beams also involved distortional and local buckling, although the magnitudes of local deformations were observed to be small and less visible due to the extensive lateral rotation of the beam.

Conclusions

This paper presented a numerical study on the structural behavior of sigma-shaped CFS beams subjected to different restraint conditions. The FE models developed were presented and validated with experimental results in terms of the maximum axial restraint forces, vertical displacements, critical temperatures and times and failure modes.

The good agreement between the experimental and numerical results confirms that the FE models developed are suitable to predict the structural behavior of CFS beams under fire conditions. However, the numerical critical temperatures were slightly lower than the experimental ones in the simply supported and axially restrained beams and the opposite in the axially and rotationally restrained beams, this for both sigma and 2-sigma section beams. For more, the numerical failure modes were very similar with the experimental ones.

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