THE STRUCTURAL BEHAVIOUR IN FIRE OF A COLD-FORMED STEEL PORTAL FRAME HAVING SEMI-RIGID JOINTS

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Abstract

This paper describes a non-linear finite element study into the effects of elevated temperature on a cold-formed steel portal frame having semi-rigid joints. Numerical modelling was carried out using ABAQUS finite element analysis software with shell elements used to capture localised buckling effects. Results for the ambient shell models are compared against previous full-scale tests. Material properties are taken from the literature, in order to predict the behaviour of the frame at elevated temperature. The results of finite element beam models are compared against those of shell models to enable comparison. At elevated temperature, shell models are shown to detect failure much earlier within the fire. Therefore shell models are recommended for such studies, for a conservative approach.

Keywords: cold-formed steel, fire, portal frame, semi-rigid joints, finite element analysis.

INTRODUCTION

Cold-formed steel portal frames can be a viable alternative to conventional hot-rolled steel portal frames for commercial, industrial and agricultural buildings with spans up to 20 m (Lim and Nethercot, 2004). Despite this, research on the structural behaviour of cold-formed steel portal frames at elevated temperature remains limited. Further research into analysis methods and the collapse mechanism is required, in order to protect fire authorities, persons and adjacent buildings in close proximity to the structure.
Research into the behaviour of hot-rolled steel portal frames at elevated temperature has been carried out by a number of researchers investigating experimental and finite element beam models (Song et al, 2009, Rahman et al, 2011). The Steel Construction Institute (SCI) P313 guidance document (Simms and Newman 2002) outlines the design for hot-rolled steel portal frames in fire boundary conditions. There is no such guidance for structural engineers related to the design of cold-formed steel portal frames in fire boundary conditions.
This paper describes a study of the structural behaviour of a cold-formed steel portal frame at ambient and elevated temperature. Numerical modelling of the frame was carried out using ABAQUS finite element analysis software. The results under loading at ambient temperature were validated against ambient full-scale tests and numerical modelling found in the literature. In order to accurately capture localised buckling effects, shell elements were used to model the back-to-back cold-formed steel members. Spring elements were included to idealise the effects of bolt-hole elongation. Lateral restraint to the frame was provided at both the purlin and side rail locations. The material properties at elevated temperature were taken from literature (Chen & Young, 2004). An additional study was carried out using beam elements, to enable the effect of elevated temperature upon the structural behaviour to be compared. For this preliminary study, initial imperfections were not included within the finite element modelling and columns were assumed as unprotected.

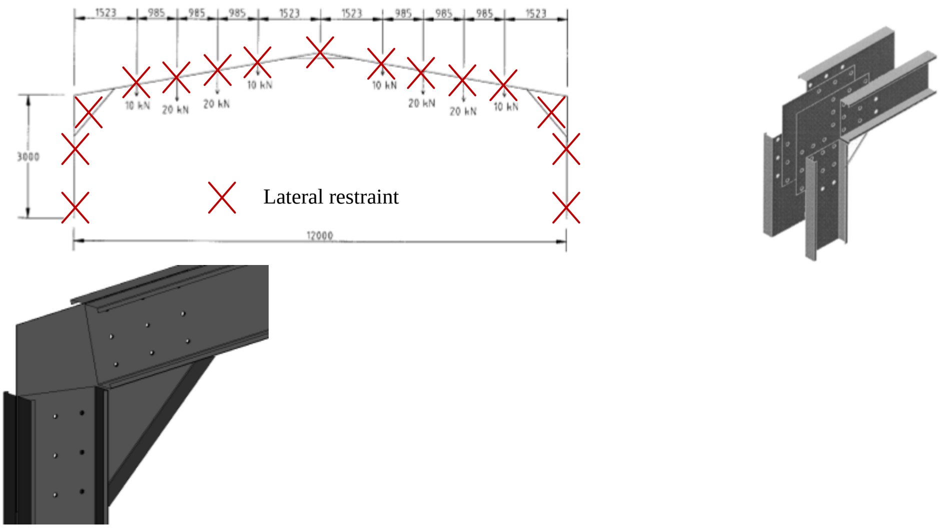
1 LITERATURE REVIEW

Previous research investigating cold-formed steel portal frames at elevated temperature tested experimental and numerical models of a frame with modest span (Pyl et al. 2012). The site fire test showed inwards collapse behaviour of the frame with sigma cold-formed steel sections used for the primary load bearing members. In the subsequent SAFIR finite element work, beam elements were used to model the sections with attention made to girders, columns, roof purlins and wall girts only. Experimental and numerical modelling research at ambient temperature (Lim and Nethercot, 2004; Jackson et al., 2012; Wrzesien et al, 2012) have demonstrated the importance of taking the effects of semi-rigid joints into consideration. Recent research indicated that elevated temperature can significantly affect the behaviour of cold-formed steel joints. This is not only in terms of the moment capacity of channel sections in vicinity of the bolt-group, but also in terms of the bearing capacity of the bolt holes (Lim and Young, 2007).

2 PORTAL FRAME FINITE ELEMENT MODELS

2.1 Structure details

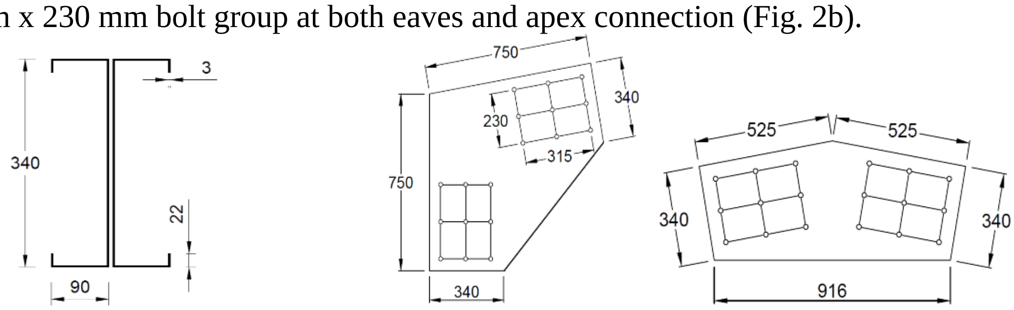
Fig. 1a details the geometry of the frame including the locations of lateral restraint. Fig. 1b shows the typical eaves connection detail used in cold-formed steel portal frame construction.



a) Geometry of portal frame b) Geometry of connections

Fig. 1 Geometry of portal frame structure

The frame is formed from bolted back to back channel sections through two 3 mm steel plates (Fig. 2a). A 12 m span frame, with 3 m eaves height and a 10º pitch was considered, using a 315 mm x 230 mm bolt group at both eaves and apex connection (Fig. 2b).



a) Back-to-back section dimensions b) Frame A bolt group on eaves and apex brackets

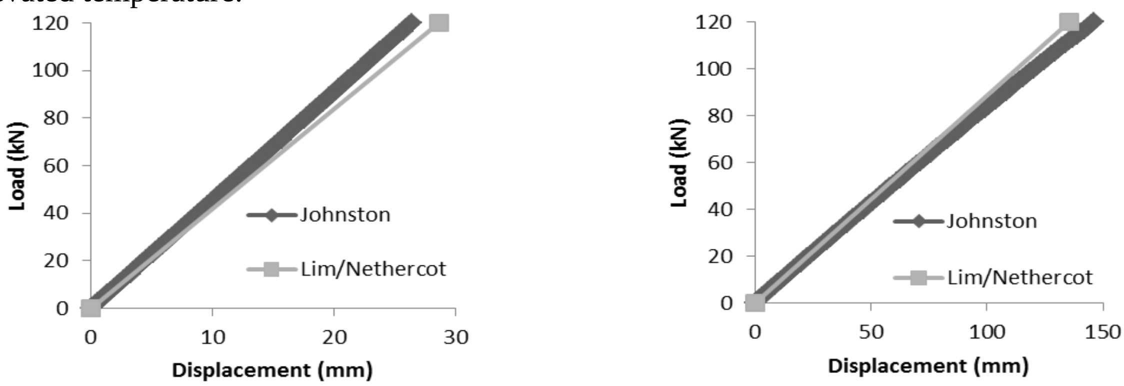
Fig. 2 Details of the frame (mm)

2.2 Numerical Modelling

According to Eurocode 3, advanced calculation models (such as the one presented in this paper) may be used for the design of Class 4 sections when all stability effects are taken into account. Therefore, for numerical calculation carried out using the finite element method, shell elements should be used to accurately capture local buckling (Franssen & Real, 2010). A non-linear static riks, elastic-perfectly-plastic model was composed using the finite element package ABAQUS, with S4R (4-node, reduced integration) shell elements. For the preliminary investigation presented in this paper, initial imperfections were not modelled. At ambient temperature, the following material values were used: Young's Modulus, E = 210,000 N/mm^2 and Yield Stress = 515 N/mm^2. These were subsequently altered to represent the reduced strength properties of cold-formed steel at elevated temperature. In order to represent the semi-rigid joints, spring elements were used to represent bolt-hole elongation with a stiffness of 10580 N in the y and z directions.

2.3 Ambient Temperature

A finite element shell model was created at ambient temperature and validated using published literature (Lim and Nethercot, 2004). The column bases were treated as pinned supports. The deflection at the apex and eaves levels was compared against an applied load of 120 kN. From Fig. 3, it can be seen that the full span ambient frame model shows good agreement with the published literature. Tab. 1 breaks down the exact values, with 7.7% and 8.2% difference in lateral spread at eaves level and vertical apex deflection respectively. This variance can be explained, in part, by the redistribution of forces within the frame. This enabled suitable validation for the preliminary investigation of the frame's behaviour at elevated temperature.



a) Validation at eaves b) Validation at apex

Fig. 3 Validation of finite element shell model at ambient temperature

Tab. 1 Displacement at 120 kN load at ambient temperature

	Lim/ Nethercot (mm)	**Ambient frame model (mm)**	**Difference (mm) [(Lim/Nethercot) - Ambient frame model]**	**Percentage difference (%)**
Lateral spread at eaves	28.6	26.4	2.2	7.7
Vertical at apex	135.2	146.3	-11.1	-8.2

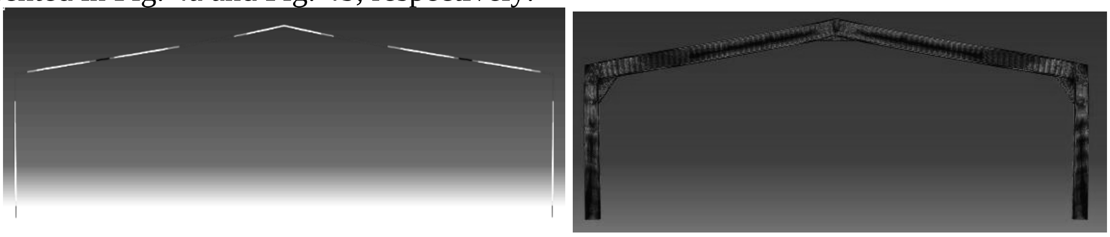
2.4 Elevated Temperature

For initial investigation at elevated temperature, a static approach was taken. Models were analysed, with the material stiffness altered for each temperature increment. For each increment, temperature was kept uniform across the entire structure, with the temperature difference between the hot gas and steel section assumed to be negligible for the thin cold-formed steel members. A total load of 120 kN was applied, initially at 250°C, up to a maximum temperature increment of 700°C. For the semi-rigid joint shell models, eaves and apex brackets were treated as fully restrained. Tab. 2 summarises the material properties used within the models.

Tab. 2 Material properties used in analysis (Chen and Young, 2004)

Steel temperature (°C)	22	250	400	450	500	550	600	700
Young's Modulus E (N/mm²)	210000	171696	146496	138096	100609	68632	41427	16200
Yield Stress fy (N/mm²)	515	494	454	409	347	267	170	48.9

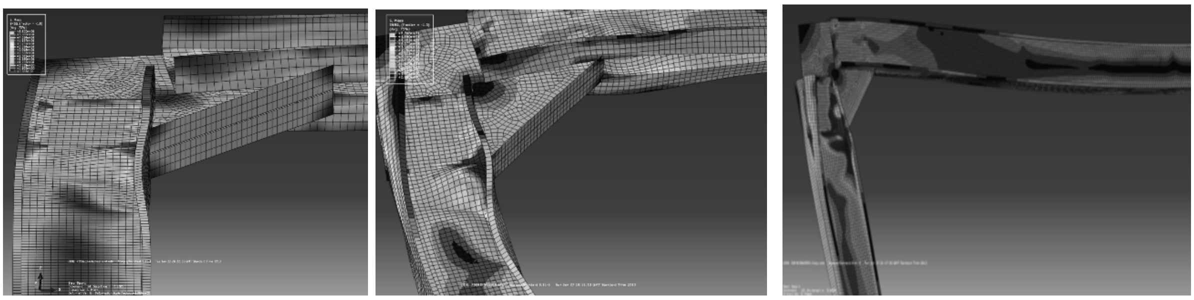
The ABAQUS beam and shell models of the cold-formed steel portal frame at 250°C are presented in Fig. 4a and Fig. 4b, respectively.



a) Beam Model b) Shell Model

Fig. 4 ABAQUS Beam model and shell model of the cold-formed steel portal frame at 250°C

Fig. 5a and Fig. 5b show the buckling of the shell model, at 550°C and 700°C, respectively. It can be seen that the buckling failure occurs below the eaves bracket, where the stiffness is greatly reduced, through a combination of coupled instability modes. As the brackets were fully restrained, they do not experience buckling, forcing the frame to fail through the channel sections. Fig. 5c demonstrates the behaviour of the column and rafter at 700°C.



a) Failure at 550°C b) Failure at 700°C c) Failure at 700°C

Fig. 5 Failure at eaves bracket connection

3 RESULTS

3.1 Shell Models

The results from the ABAQUS finite element shell models demonstrate the high sensitivity of cold-formed steel structures at elevated temperature. Fig. 6 shows the total load carrying capacity of the structure against the mid-span apex displacement for each respective temperature considered.

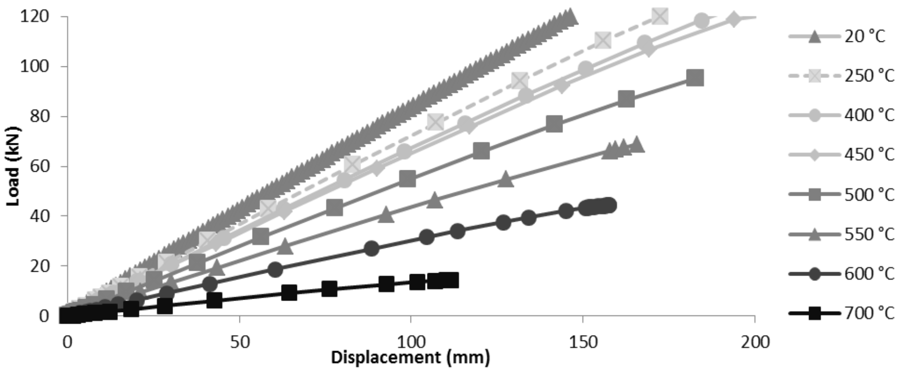


Fig. 6 Load carrying capacity of the structure against apex displacement per temperature interval

From Fig. 6, it can be seen that up to 450°C, the structure can carry a specified load up to 120 kN load. However at 500°C, the structure was only capable of carrying 79% of the specified load (95.4 kN). At 550°C and 600°C, the load carrying capacity was reduced to 57% and 37%, respectively. At 700°C, the highest temperature considered in this study, the structure was only capable of carrying 12% of the specified load (14.4 kN). Fig. 7 illustrates the apex displacement of the structure as a function of the increase in temperature. As the structure is unable to take the full load at temperatures exceeding 450°C, it is compared using a load equal to 10% of the total specified load of 120 kN (equal to 12 kN). From Fig. 7, it can be seen that between 600°C to 700°C, the rafter undergoes the largest relative displacement.

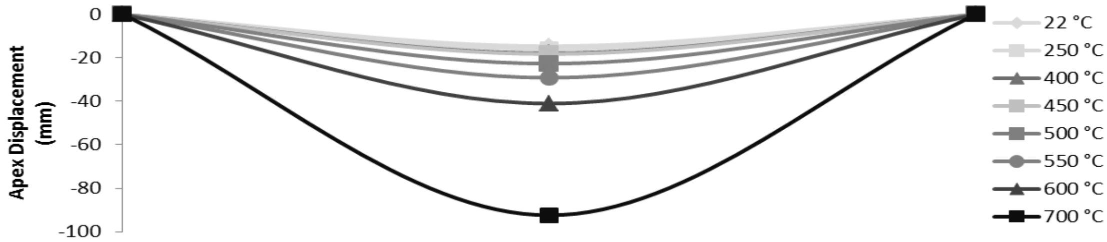


Fig. 7 Apex displacement, per temperature increment for 10% of the total load

The performance of the shell models under loading were compared using the ISO 834 Standard and Hydrocarbon nominal temperature-time curves. Fig. 8 demonstrates the load carrying capacity of the shell model, with respect to the nominal temperature-time curves. For the ISO 834 Standard curve, the structure loses its capacity to carry the specified load of 120 kN between 2-3 minutes (between 400-500°C). For the case of the Hydrocarbon fire curve, the structure loses its capacity to carry the specified load within 1 minute.

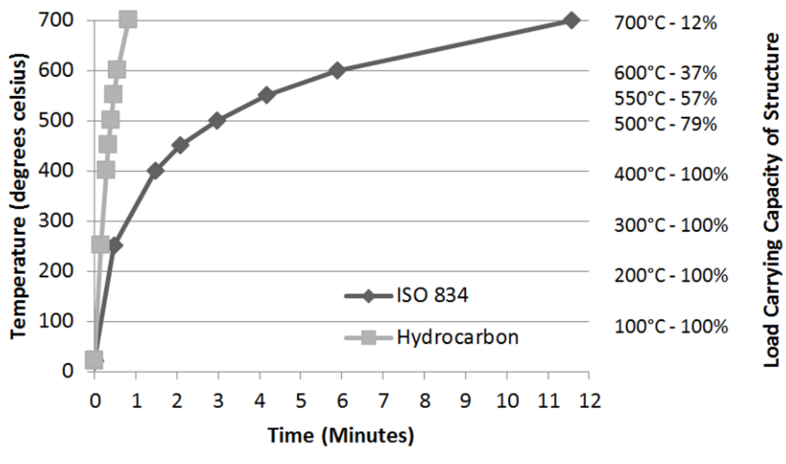


Fig. 8 Load carrying capacity of the shell model with respect to nominal temperature-time curve.

3.2 Comparison between shell and beam models

Tab. 3 outlines the comparison between beam and shell models for the temperature range 22°C to 700°C. The shell models show a higher sensitivity to temperature between 450-550°C. At 500°C, the beam model is unable to predict the failure, detected by the shell model. The largest difference in load carrying capacity is 40.8% at 550°C, whereas at 700°C the difference is reduced to 7.8%.

Tab. 3 Comparison between shell and beam models

Temp	**Load carrying capacity (up to 120 kN)**			**Apex displacement (mm)[1]**		**Eaves displacement (mm)[1]**	
(°C)	**Shell** (%)	**Beam** (%)	**Difference (B-S)** (%)	**Shell**	**Beam**	**Shell**	**Beam**
22	100.0	100.0	0.0	14.6	6.4	2.6	1.1
250	100.0	100.0	0.0	16.0	7.8	3.2	1.3
400	100.0	100.0	0.0	17.9	9.2	3.5	1.5
450	100.0	100.0	0.0	18.2	9.7	3.7	1.6
500	79.5	100.0	20.5	22.7	13.4	5.0	2.3
550	57.2	98.0	40.8	29.2	19.6	5.5	3.3
600	36.9	61.0	24.1	41.0	32.7	8.0	5.4
700	12.0	19.9	7.8	92.4	85.3	18.7	13.7

Note: [1] To enable comparison, apex and eaves displacement is represented at a load 10% of the total load.

4 SUMMARY

This paper has described a preliminary numerical study of a cold-formed steel portal frame, having semi-rigid joints, at elevated temperature. A static finite element numerical analysis has been performed on a loaded frame up to a maximum temperature of 700°C. It should be noted that within this initial study, the stiffness value of the springs which idealize bolt hole elongation have been kept constant for each of the shell models.

The study has demonstrated that beam models are not capable of predicting the same load carrying capacity and displacement when compared to shell models. At elevated temperature, shell models are shown to detect failure much earlier within the fire.

Therefore shell models are recommended for such studies, for a conservative approach. It is suggested that future research investigates a multi-bay portal frame arrangement, using a dynamic analysis which incorporates initial imperfections and actual bolt representation.

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