DISTORTIONAL BUCKLING BEHAVIOUR OF FIRE EXPOSED COLD-FORMED STEEL COLUMNS

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ABSTRACT

Cold-formed steel sections are commonly used in low-rise commercial and residential buildings. During fire events, cold-formed steel structural elements in these buildings can be exposed to elevated temperatures. Hence after such events there is a need to evaluate their residual strengths. However, only limited information is available in relation to the residual strength of fire exposed cold-formed steel sections. This research is aimed at investigating the distortional buckling capacities of fire exposed cold-formed lipped channel sections. A series of compression tests of fire exposed, short lipped channel columns made of varying steel grades and thicknesses was undertaken in this research. Test columns were first exposed to different elevated temperatures up to 800 ^{o}C, and then tested to failure after cooling down. Suitable finite element models were developed with post-fire mechanical properties to simulate the behaviour of tested columns and were validated using test results. The residual compression capacities of short columns were also predicted using the current cold-formed steel standards and compared with test and finite element analysis results. This comparison showed that ambient temperature design rules for columns can be used to predict the residual compression capacities of fire exposed short or laterally restrained cold-formed steel columns provided the maximum temperature experienced by the column can be estimated after a fire event. Such residual capacity assessments will allow engineers to evaluate the safety of fire exposed buildings. This paper presents the details of this experimental study, finite element analyses and the results.

KEYWORDS

Cold-formed steel, Fire exposed steel columns, Post-fire, Residual strength, Distortional buckling.

INTRODUCTION

Thin-walled and cold-formed steel columns are susceptible to various buckling modes (local, distortional, flexural and flexural-torsional) and their interactions. These buckling behaviours have been investigated in the past at both ambient and elevated temperatures using both experimental and numerical studies (Feng et al. 2003, Chen and Young 2007, Ranawaka and Mahendran 2010, Gunalan and Mahendran 2013). They showed that ambient temperature design methods can be used to predict the elevated temperature strengths of cold-formed steel columns reasonably well provided appropriately reduced mechanical properties are used in the strength calculations. In practical applications, cold-formed steel studs are likely to be protected by plasterboards and insulations. Following fire events, the gypsum plasterboards can be easily removed from the wall frames to inspect the damage caused by fire. The structural engineer has to then decide if the strength of the light gauge steel wall frame is still adequate for future use if lined with new plasterboards. The behaviour of

structural steel members after a fire event has been investigated by Tide (1998). However, the behaviour of cold-formed steel members after a fire event has not been investigated yet. There are also no design guidelines for assessing fire exposed cold-formed steel members. As a result of this limited knowledge and understanding in this field, over-conservative decisions are likely to be made when evaluating the residual capacities of fire exposed cold-formed steel members. In the event of a fire, the section and member load bearing capacities of steel members are mostly affected by the changes in their mechanical properties of steel sections. Current design standards contain no information detailing the mechanical properties of cold-formed steels after being exposed to elevated temperatures. Recent research at the Queensland University of Technology investigated the post-fire mechanical properties of cold-formed steels (Gunalan and Mahendran 2014a) and developed suitable equations to predict the residual mechanical properties (Figure 1) after being exposed to elevated temperatures up to 800 °C. Such knowledge of the residual mechanical properties as a function of exposed temperature in a fire will allow engineers to make important decisions in relation to the re-use of fire exposed cold-formed steel structural members and frames. Their decision will be based on the assumption that the residual member capacities are simply proportional to the reduction in their mechanical properties. This may not always be true. Post-fire mechanical property studies showed that yield strength was reduced at a faster rate than elastic modulus when exposed to elevated temperatures while there was improved ductility with increasing exposed temperature. Hence there is a need to investigate the residual capacities of cold-formed steel members subject to various buckling modes such as local and distortional buckling. It is also not known whether the residual member capacities can be calculated using ambient temperature design rules with the measured post-fire mechanical properties of cold-formed steels given in Gunalan and Mahendran (2014a). Therefore a detailed study was undertaken to investigate the distortional buckling behaviour of light gauge cold-formed steel compression members after being exposed to elevated temperatures. Ambient temperature design rules were modified by including the post-fire mechanical properties and their accuracy was investigated by using the test and finite element analysis results. This paper presents the details of this investigation and the results.
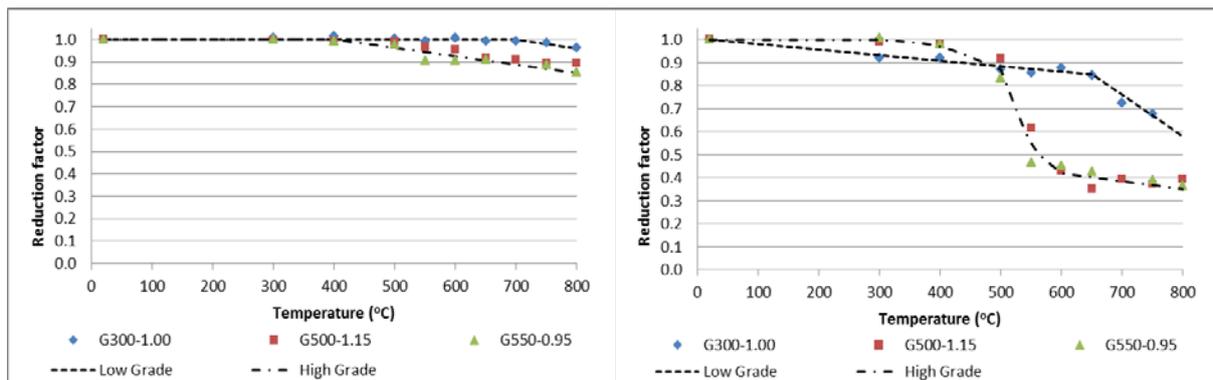
(a) Elastic modulus   (b) Yield strength
Figure 1. Mechanical property reduction factors versus exposed temperature.

EXPERIMENTAL STUDY

This experimental study was conducted to investigate the behaviour of cold-formed steel compression members subject to distortional buckling at ambient and exposed temperatures up to 800 °C. Cold-formed steel lipped channel specimens were heated up to the required temperature and then allowed to cool down at ambient temperature. An axial compressive load was applied thereafter at a constant rate until failure. The lipped channel sections used here were 40mm x 40mm x 5mm with thicknesses of 0.95, 1.0 and 1.15 mm and a clear column height of 300 mm. Both low (G300-1.00 mm) and high grades (G500-1.15 mm and G550-0.95 mm) were used. Preliminary analyses were undertaken to ensure that the critical failure mode was distortional. A total of nine steel columns from each grade and thickness were tested at ambient temperature and after being exposed to eight different elevated temperatures. All the compression tests were carried out using fixed-end supports (Figure 2(a)). The ultimate loads of different steel grade specimens are compared for ambient and exposed temperatures

in Table 1, which shows that the ultimate loads decrease with increasing exposed temperatures. Although a slight increase in ultimate loads occurred, this was considered to be due to experimental variations within a reasonable range. Figures 2 (b) and (c) show the typical distortional buckling failure mode and axial compression load versus deflection curves.
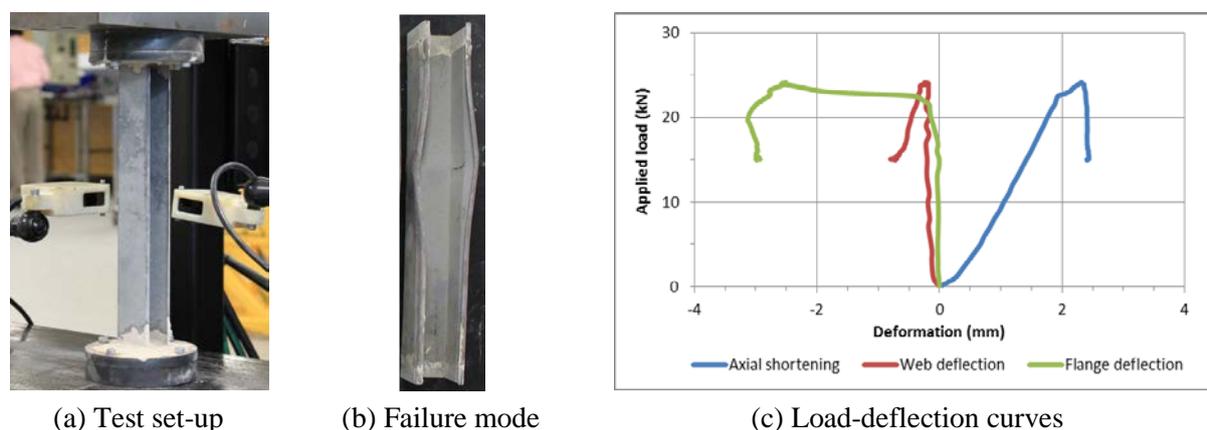

    (a) Test set-up   (b) Failure mode   (c) Load-deflection curves
Figure 2. Test set-up and results for G300-1.00 specimen for an exposed temperature of 800 ^{o}C.

Table 1. Comparison of ultimate loads from tests, FEA and design standards

Temp. (^{o}C)	G300 - 1.00 mm				G500 - 1.15 mm				G550 - 0.95 mm			
	Test P_{ult} (kN)	Test/ FEA	Test/ AS 4600	Test/ DSM	Test P_{ult} (kN)	Test/ FEA	Test/ AS 4600	Test/ DSM	Test P_{ult} (kN)	Test/ FEA	Test/ AS 4600	Test/ DSM
20	30.8	0.93	0.95	0.96	54.2	0.90	1.03	0.99	38.1	0.91	0.99	0.91
400	35.3	1.09	1.13	1.13	48.9	0.89	0.91	0.88	41.5	1.01	1.11	1.02
500	36.2	1.13	1.19	1.18	48.4	0.92	0.94	0.92	33.2	0.93	1.00	0.93
550	31.8	1.03	1.08	1.08	44.2	1.02	1.01	1.01	25.7	0.91	0.93	0.93
600	31.6	1.09	1.08	1.08	38.8	1.08	1.14	1.11	29.0	1.08	1.08	1.08
650	28.6	1.01	0.99	0.99	35.9	1.15	1.21	1.14	25.9	0.97	1.01	1.01
700	29.5	1.13	1.11	1.08	37.9	1.09	1.16	1.11	-	-	-	-
750	22.6	0.92	0.91	0.88	31.9	1.00	1.06	1.01	29.2	1.09	1.17	1.14
800	24.0	1.04	1.10	1.05	34.9	1.08	1.14	1.10	24.9	1.09	1.08	1.07
Mean		1.041	1.060	1.048		1.014	1.067	1.030		0.999	1.046	1.011
COV		0.075	0.086	0.087		0.092	0.097	0.089		0.080	0.074	0.082

FINITE ELEMENT MODELLING

In this research ABAQUS was used in the finite element analyses (FEA) of fire exposed cold-formed steel compression members subject to distortional buckling. S4 element type with an element size of 3 mm x 3 mm was used based on the convergence study. Due to the symmetric conditions of test specimen and loading, a half-length model was used with appropriate boundary conditions as shown in Figure 3(a). An elastic-perfect plastic material model was used. The measured ambient and post-fire temperature mechanical properties of G300-1.00, G500-1.15 and G550-0.95 mm thick cold-formed steels from Gunalan and Mahendran (2014a) were used in the analyses. Elastic buckling analyses were used to determine the elastic buckling loads and modes while nonlinear analyses were used to determine the ultimate loads and deformations. In this research the section thickness (*t*) was used as the imperfection value for thin cold-formed lipped channels. The ultimate loads from FEA were then compared with test results. Table 1 presents the mean values and the associated coefficient of variations for the ratio of ultimate loads from test and FEA, which indicates a reasonably good agreement between test and FEA results. Accuracy of the developed models was also assessed by

comparing the failure modes from FEA and tests. Figures 2(b) and 3(b) show that the failure modes from test and FEA are similar, ie. distortional buckling mode. Hence it can be concluded that the developed finite element models are able to accurately simulate the distortional buckling behaviour of cold-formed steel compression members exposed to elevated temperatures.

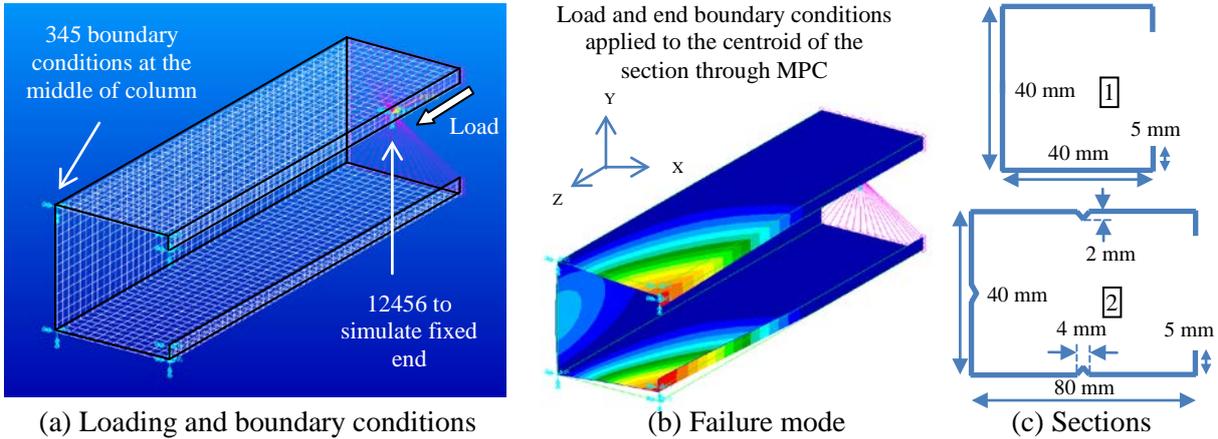| (a) Loading and boundary conditions | (b) Failure mode | (c) Sections |

Figure 3. Finite element analyses

The ultimate load results from tests and FEA are also compared using a non-dimensional format in Figure 4(a), where the non-dimensional slenderness λ is defined as $\sqrt{f_{yT}/f_{crT}}$ where f_{yT} and f_{crT} are the yield and elastic buckling strengths, respectively, and P_y is the squash load of column at exposed temperature T.

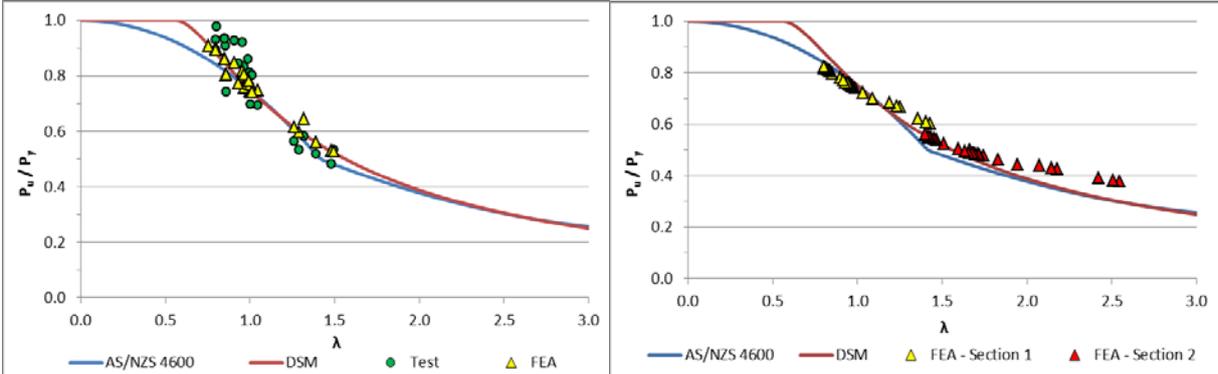| (a) Test and FEA validation | (b) Parametric study |

Figure 4. Comparison of test and FEA results with code predictions

DESIGN RULES

In this section, the ultimate loads obtained from tests and FEA are compared with the predictions from the available design equations for cold-formed steel structures (AS/NZS 4600, AISI S100 and Direct Strength Method (DSM) (Schafer 2008)) using the measured post-fire mechanical properties. The statistical model recommended by AISI S100 (2007) was used to determine the capacity reduction factors for the values obtained from the current design rules. AS/NZS 4600 (2005) recommends a capacity reduction factor of 0.85 for compression members. AS/NZS 4600 and AISI S100 give identical design rules for distortional buckling and hence only AS/NZS 4600 design rules are discussed in this paper. The DSM is an alternative methods to determine the distortional buckling strength of cold-formed steel members. Table 1 and Figure 4(a) show that test and FEA results agreed well with AS/NZS 4600 and DSM predictions for fire exposed columns subject to distortional buckling. Therefore it can be concluded that AS/NZS 4600 design rules for distortional buckling can be conservatively used to predict the ultimate loads of fire exposed lipped channel sections.

The accuracy of these design rules for slender columns was further investigated in a FEA based parametric study. Two sections shown in Figure 3(c) including Section 1 that was used in our tests were considered in the parametric study with G300-1.00 mm, G500-1.15 mm and G550-0.95 steels. Small stiffeners were used in the web and flange elements of Section 2 to eliminate local buckling. Three steel grades and thicknesses (G300-1.00, G500-1.15 and G550-0.95) and eight different exposed temperatures (400, 500, 550, 600, 650, 700, 750 and 800 ^{o}C) were considered. In the analyses the nominal yield strengths (300, 500 and 550 MPa) were used with an elastic modulus of 200 GPa at ambient temperature. Post-fire mechanical properties were predicted using Gunalan and Mahendran's (2014a) proposed equations. Figure 5(b) shows that these FEA results closely agreed with AS/NZS 4600 and DSM predictions for fire exposed slender columns subject to distortional buckling.

ESTIMATION OF RESIDUAL COLUMN STRENGTH

Figures 5(a) and (b) show the variation of the residual ultimate load ratio with respect to different exposed temperatures based on the FEA results obtained from the parametric study for Sections 1 and 2 (Figure 4(c)). The reduction in ultimate load follows a similar trend to that of the reduction in yield strength. This is expected as the reduction in elastic modulus is less than 15% even when exposed to 800 ^{o}C for low and high grade steels (Figure 1(a)). The reduction in ultimate load was less than the reduction in yield strength for both low and high grade steels as shown in Figures 5(a) and (b).

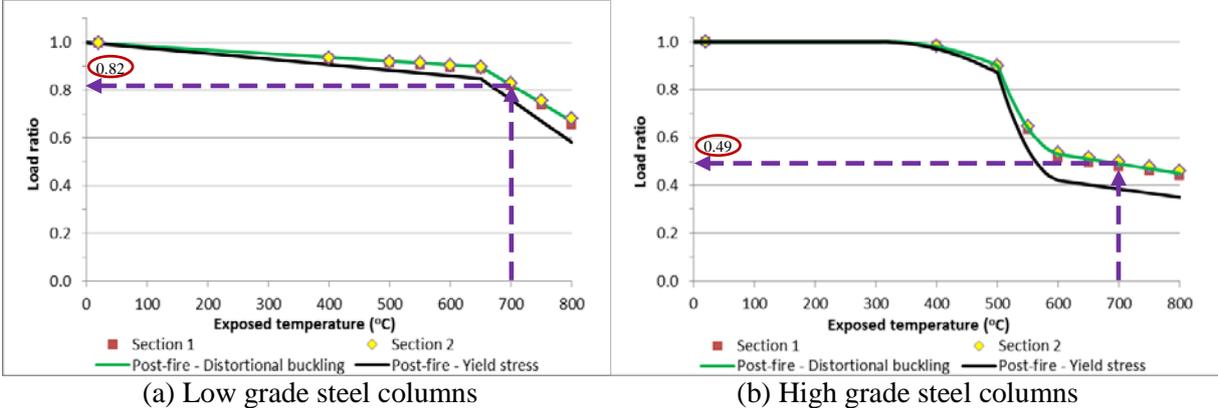(a) Low grade steel columns   (b) High grade steel columns
Figure 5. Simplified method to find the residual capacity

This study has shown that cold-formed steel columns can regain 90% of the original distortional buckling capacities if they are exposed to temperatures below 500 ^{o}C. If the exposed temperature of a column in a fire is known, then Figures 5(a) and (b) can be used to obtain its residual capacity. For example, the load ratios of low and high grade steel columns exposed to a temperature of 700 ^{o}C were predicted as 0.82 and 0.49 as shown in the figures. Alternatively AS/NZS 4600 and DSM design rules with appropriately reduced post-fire mechanical properties can be used to predict the residual capacities of cold-formed steel columns if the maximum exposed temperature can be estimated.

RECOVERY OF DISTORTIONAL BUCKLING CAPACITIES

Figures 6(a) and (b) show the variation of residual ultimate load ratios at elevated temperatures and after cooling down for low and high grade steel sections, subject to distortional buckling. The FEA results of lipped channel columns at elevated temperatures from Ranawaka and Mahendran (2010) and our research results are plotted in these figures. Figures 6(a) and (b) clearly indicate the recovery of compression capacity of cold-formed steel members after they cool down to ambient temperature. The regain of strength increased with higher temperatures and the maximum regain of 60% and 40% were obtained for low and high grade steel columns, respectively. The regain of column strength was higher for low grade steel columns compared to high grade steel columns when they cool down to ambient temperatures. Figures 6(a) and (b) also show that the capacity of cold-formed steel column is significantly reduced when it is exposed to elevated temperatures in a fire (40% at 500 ^{o}C for high grade steel columns). This may be still adequate for a fire limit state as the fire design load is a fraction

of ambient temperature design load. In contrast, in the case of fire exposed cold-formed steel columns, the residual column capacity should not be reduced significantly as they need to be able to support the higher ambient temperature design loads. This appears to be satisfactory as both low and high grade steel columns retain 90% of their capacities if the exposed temperature is below 500 $^\circ$C.

(a) Low grade steel columns   (b) High grade steel columns
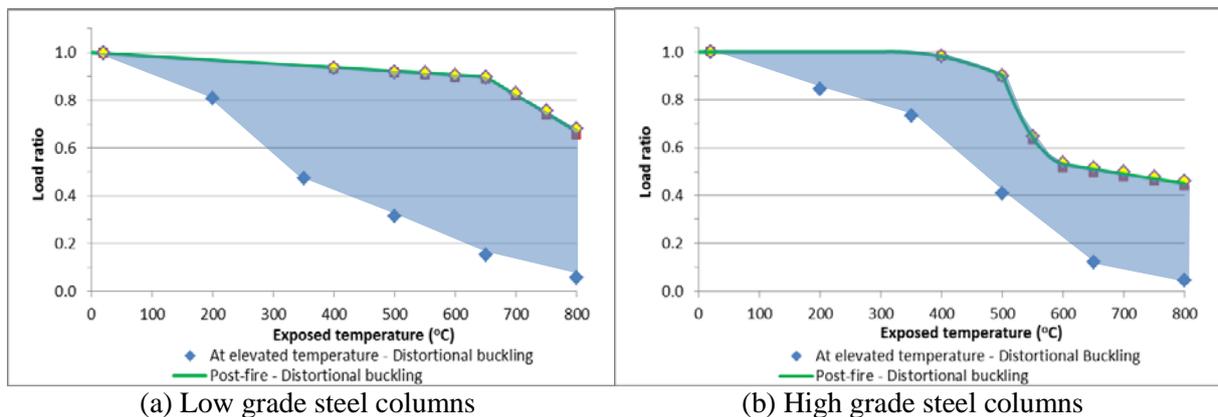
Figure 6. Recovered capacity regime

CONCLUSIONS

This paper has described an investigation into the residual distortional buckling capacities of cold-formed steel columns after being exposed to elevated temperatures. Twenty seven tests were conducted on cold-formed steel lipped channel sections made of different grades and thicknesses exposed to temperatures varying from 20-800 $^\circ$C. Suitable finite element models of tested columns were also developed and validated using test results. It was found that cold-formed steel columns were able to regain 90% of their original distortional buckling capacities if the exposed temperature was below 500 $^\circ$C. The reduction in ultimate capacity followed a similar trend to that of yield strength and hence post-fire mechanical properties alone can be used to predict the residual capacities conservatively. However, the reduction in ultimate capacities was less severe than that predicted by yield strength reduction factors for both low and high grade steel columns. A detailed comparison of ultimate capacity results with those predicted by the current ambient temperature design rules showed that these equations can also be used to predict the residual compression capacities reasonably well when appropriately reduced post-fire mechanical properties are used.

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