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# Effects of elevated temperatures on ultimate moment capacity of bolted moment-connections between cold-formed steel members

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## Abstract

Experimental investigations at ambient temperature into the behaviour of bolted moment-connections between cold-formed steel members have previously been described. Full-scale joint tests have demonstrated that the channel-sections being connected are susceptible to premature failure, the result of web buckling caused by the concentration of load transfer from the bolts. The results of non-linear elasto-plastic finite element analyses have been shown to have good agreement. No consideration, however, has been given to the behaviour of such connections at elevated temperatures. This paper describes non-linear elasto-plastic finite element parametric studies into the effects of elevated temperatures on bolted moment-connections between cold-formed steel members; simple design rules are proposed that will enable designers to take into account the effects of elevated temperatures.

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**Introduction**

Bolted moment-connections between cold-formed steel members, formed through brackets bolted to the webs of the cold-formed steel sections being connected (see Fig.1), are used for the joints of portal frames [Ref. 1, 2], multi-storey frames [Ref. 3, 4], and racking systems [Ref. 5]. The behaviour of such joints, however, has only been considered at ambient temperature, with no consideration being given to joint behaviour at elevated temperatures.

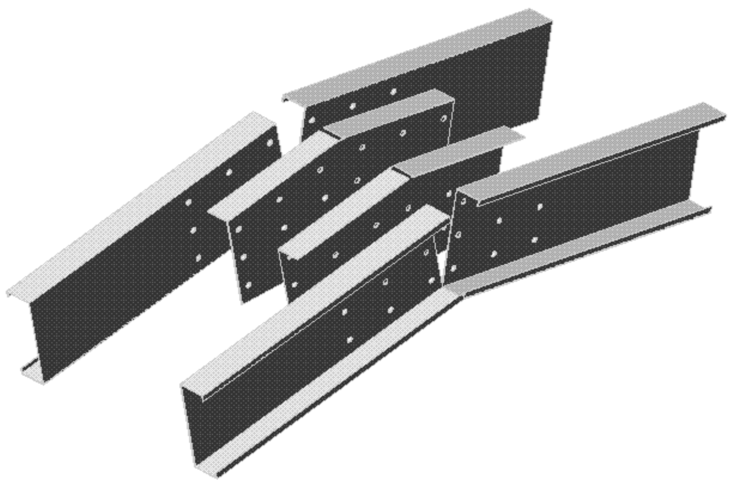


Figure 1. Details of typical bolted moment connection

The lack of design information for elevated temperatures means that it is not always possible to design safely cold-formed steel frames having such joints in fire, without resorting to the use of fire protection. Conservative design recommendations are normally based on simple modifications to the fire design rules of hot-rolled steel structures. However, in the case of hot-rolled steel, local buckling is generally not a problem. On the other hand, with cold-formed steel, both local and distortional buckling is common, making the design of these structures in fire problematic. Whilst ideally full-scale fire tests should be conducted, the cost of these tests will be prohibitive. In fact, it has only been in recent years that cold-formed steel coupon tests have been conducted at elevated temperatures.

Chen and Young recently have conducted a series of such tests [Refs 6, 7]. From the results of these tests, equations were developed that predict the stress-strain curves at elevated temperatures. In this paper, the stress-strain curves proposed by Chen and Young [Ref 6] are applied to a numerical investigation on the strength of bolted-moment connections between cold-formed steel members at elevated temperatures.

In this paper, the strength in fire of the channel-sections at the joints are investigated, which are susceptible to premature web buckling, induced by concentrated load transfer from the bolt-group. Curves are presented that illustrate how this mode of failure is affected by elevated temperatures. For cold-formed steel frames, where failure is generally non-ductile, the failure load of the joints is more important than for hot-rolled steel frames, where the frames continue to exhibit increased strength after the formation of the first plastic hinge. Simple design rules are proposed, from the results of this study, which will allow designers to take into account the effects of elevated temperatures.

**Stress-strain curves at elevated temperatures**

Fig.2 shows stress-strain curves for cold-formed steel at eight temperatures ranging from 22$^{o}$C to 700$^{o}$C. In this paper, the effects of elevated temperatures at these eight temperatures will be considered. The stress-strain curves are obtained from equations proposed by Chen and Young [Ref. 6]. Table 1 summarises the Young's modulus and yield stress, calculated from equations also proposed by Chen and Young [Ref. 7].

In this paper, the general purpose finite element program ABAQUS [Ref. 8] is used for the numerical investigations. In the numerical models, non-linear stress-strain material curves are modelled. The first part of the engineering stress-strain curve represents the elastic part up to the proportional limit stress with measured elastic modulus and Poisson's ratio. In this study, the Poisson's ratio is taken as 0.3 under fire conditions. Generally, the Poisson's ratio is assumed to be independent of temperature [Refs 9, 10]. Since the analysis of post-buckling involves large in-elastic strains, the engineering stress-strain curve has been converted to a true stress and logarithmic plastic strain curve for the different temperatures. These true stress and plastic true strain curve equations are specified in ABAQUS.

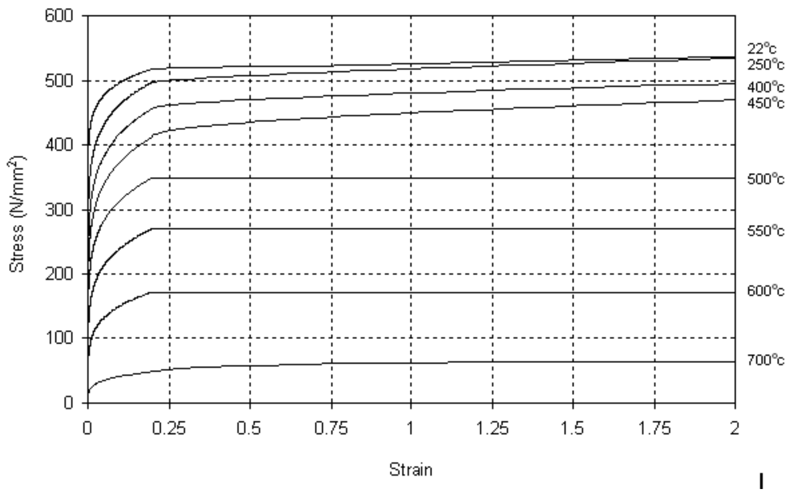



Figure 2. Stress-strain curves at different temperatures

Table 1. Summary of material properties at different temperatures

| Temp -erature T (°C) | Young's modulus E (N/mm$^2$) | Yield stress $f_y$ (N/mm$^2$) |
|---|---|---|
| 22 | 210000 | 515 |
| 250 | 171696 | 494 |
| 400 | 146496 | 454 |
| 450 | 138096 | 409 |
| 500 | 100609 | 347 |
| 550 | 68632 | 267 |
| 600 | 41427 | 170 |
| 700 | 16200 | 48.9 |

**Ultimate strength of bolted moment-connections**

Phenomenon of premature web buckling

When designing bolted moment-connections, two modes of failure can easily be prevented:

i. Overall lateral-torsional buckling of the joint. This type of buckling may be prevented through the provision of sufficient lateral restraint around the joint

ii. Buckling of the bracket. Adopting the design recommendations of Ref. 11 would ensure that the bracket has a higher moment capacity than the channel-sections being connected.

A third mode of failure, which cannot be prevented as easily, is concerned with the reduction in strength of the channel-sections at the vicinity of the joints, caused by concentrated load transfer from the bolt-group (see Fig.3). This mode of failure, referred to as premature web buckling [Ref. 12], has been observed as the governing mode of failure in a number of laboratory tests on cold-formed steel bolted moment-connections [Refs 1, 13, 14 and 15]. A full-review of all of these tests is given in Ref. 12, in which it is demonstrated some of the joints tested failed at a moment-capacity 20% lower than the calculated moment-capacity of the channel-sections.

Fig.4 shows an example of premature web buckling induced failure. As can be seen, the mode involves buckling of the web of the channel, accompanied by sympathetic flange distortion. While the resulting failure mode shape is similar to distortional buckling, the mode of failure is initiated by premature web buckling. Premature web buckling is not covered by BS5950: Part 5 [Ref. 16], or any of the other codes of practice.

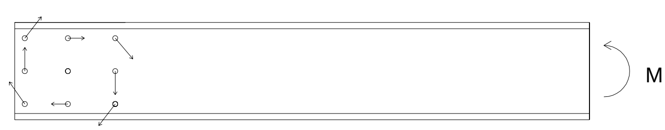



Figure 3. Free body diagram of channel-section when joint is in pure bending

Figure 4. Typical web buckling induced failure [Ref. 14]

Ref 12 describes a combination of laboratory tests and finite element analyses used to investigate this mode of failure. However, while good agreement was demonstrated between the measured ultimate moment-capacity and that predicted by using the finite element analyses, the study was only concerned with the behaviour at room temperature. In this Section, a numerical study on the influence of elevated temperatures on premature web buckling is described.

**Finite element model**

Details of the finite element model used to investigate premature web buckling in Ref. 12 are shown in Fig.5. As can be seen, the model consists of only a single channel-section loaded under pure bending. The parameters used to describe the dimensions of the bolt group are shown in Fig.5. A full description of the model is given in Ref. 12.

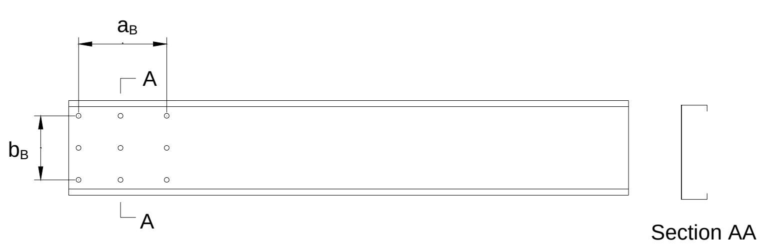



Figure 5. Details of parameters used to describe the bolt-group array

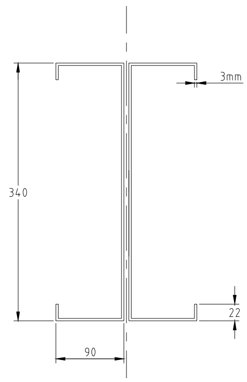



Figure 6. Dimensions of channel-section used in parametric study

**Parametric study**

A parametric study was conducted to investigate the effects of elevated temperatures on premature web buckling. Fig.6 shows the dimensions of the channel-section used for the purposes of the parametric study; the same dimension of channel-section was also used in Ref. 12.

In the parametric study described in this Section, the thickness of channel-section was varied between 2 mm and 8 mm. Three bolt-group lengths ($a_B$) were considered, namely, 200 mm, 500 mm and 1000 mm.

Fig.7(a) shows the reduction in moment capacity $M_{u,T}/M_{u,normal}$ for a bolt-group length of 200 mm. For each temperature, the values of Young's modulus, yield stress and ultimate stress are also normalised their respective values at room temperature.

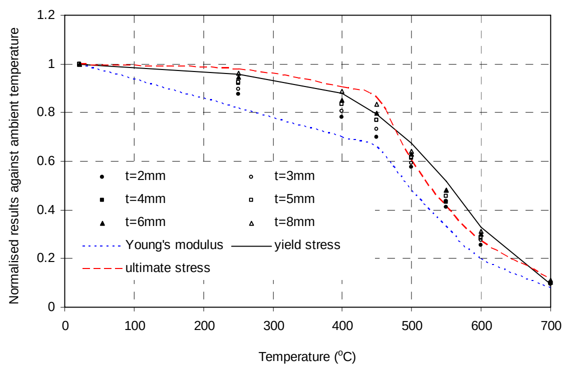
From Fig.7(a), it can be seen that the value of $M_{u,T}/M_{u,normal}$ decreases with the thickness of the channel-section. This is to be expected since plate buckling is a function of the thickness cubed. For the channel-section of thickness 8 mm, the reduction in moment capacity closely follows that of the reduction in yield stress ($f_{y,T}/f_{y,normal}$). However, for the channel-section of thickness 2 mm, the value of $M_{u,T}/M_{u,normal}$ is much lower. For example, in the case of a temperature of 400$^o$C, the reduction in strength of the 2 mm channel-section is 13% lower than that of the 8 mm section. A lower bound to the reduction in strength can be seen to be $E_T/E_{normal}$.

It is interesting to note from Fig.7(a) that the greatest variation in $M_{u,T}/M_{u,normal}$ between the different thicknesses of channel-section is 13% and occurs at a temperature 450$^o$C. At this temperature, the difference between $E_T/E_{normal}$ and $f_{y,T}/f_{y,normal}$ is also 13%. Similarly, at a temperature of 600$^o$C the variation in $M_{u,T}/M_{u,normal}$ between the different thicknesses of channel-section is 6%. At this temperature, the difference between the same material properties is also 6%.

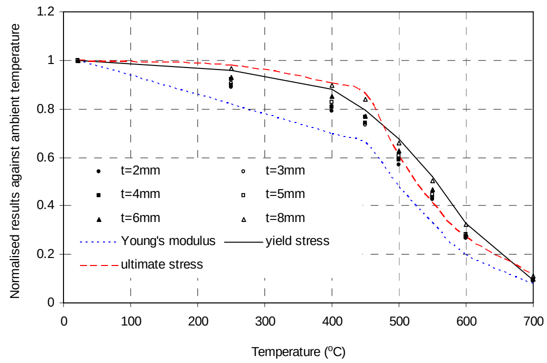
Since comparisons of the roundness of the stress-strain curves at these temperatures show no noticeable difference, it may therefore be concluded that the value of $M_{u,T}/M_{u,normal}$ is a function of both $E_T/E_{normal}$ and $f_{y,T}/f_{y,normal}$.

Fig.7(b) and Fig.7(c) show the same results for values of $a_B$ = 500 mm and 1000 mm, respectively. As can be seen, the same general trends as for Fig.7(a) can be observed, even though the value of $a_B$ has been increased significantly from 200 mm to 1000 mm. In Ref. 12, the effect of increasing $a_B$ from 200 mm to 1000 mm resulted in a 20% increase in moment capacity. While this increase in moment capacity has been taken into account when comparing the curves of Fig.9, owing to the fact that normalised results are presented, it is interesting to note that the results have no additional sensitivity to the value of $a_B$.

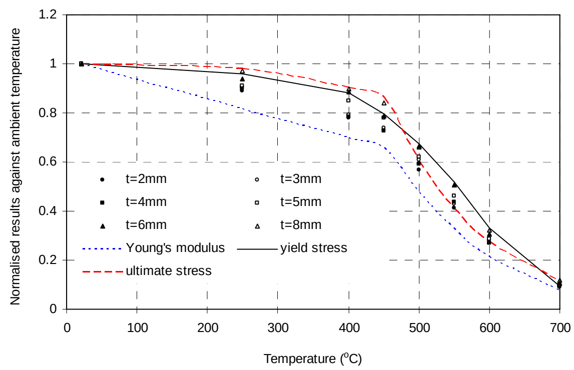
In general, as the value of $a_B$ increases, the values of $M_{u,T}/M_{u,normal}$ also increase but the range of variation between the different thicknesses of channel-sections decreases. The fact that the values of $M_{u,T}/M_{u,normal}$ are not sensitive to the value of $a_B$ is important for the design recommendations that follow.




(a) Bolt-group having $a_B$ = 200 mm




(b) Bolt-group having $a_B$ = 500 mm




(c) Bolt-group having $a_B$ = 1000 mm

Figure 7. Comparison of variation of moment capacity with material properties

## Design recommendations

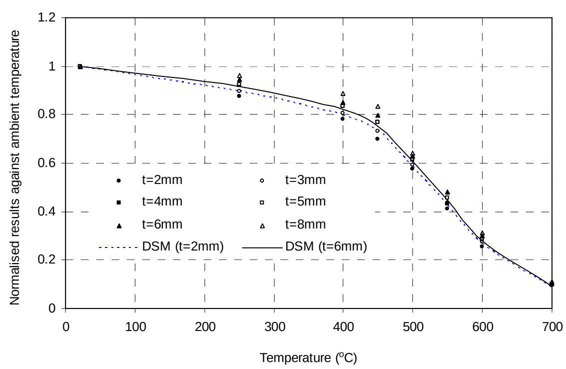
In the parametric study, it was observed that the reduction factor $M_{u,T}/M_{u,normal}$ is sensitive to both $E_T/E_{normal}$ and $f_{y,T}/f_{y,normal}$ but not to the values of $a_B$. As the moment capacity of the channel-section is a function of both $E_T$ and $f_{y,T}$, a design recommendation based on the reduction in moment capacity may be appropriate.

The direct strength method specified in the supplement to the North American Specification [Ref. 17] and the Australian/New Zealand Standard [Ref. 18] is used to predict the moment capacity of the cold-formed steel channel-sections. The nominal design strengths at elevated temperatures were calculated by substituting the reduced yield stress (0.2% proof stress) and Young's modulus into the design rules. The nominal design strengths were calculated using the cross-section and the reduced material properties as those used in the parametric study of the finite element analysis.

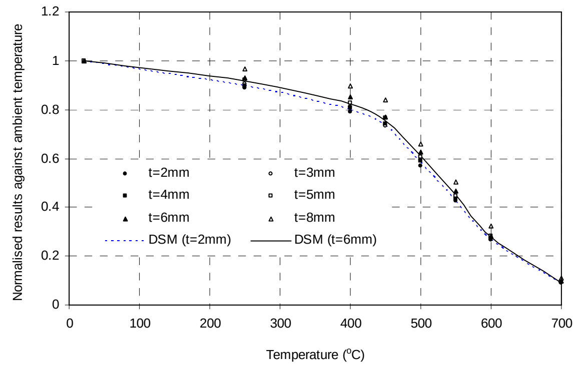
It should be noted that the current direct strength method is developed based on cold-formed steel structural members at normal room temperature by Schafer and Peköz [Ref. 19]. In this study, the direct strength method is used for cold-formed steel channel-sections subjected to bending at elevated temperatures.

Fig.8 shows the comparison of the reduction factor $M_{u,T}/M_{u,normal}$ with the reduction factor predicted using the direct strength method ($M_{DSM,T}/M_{DSM,normal}$). The value of $M_{DSM,T}/M_{DSM,normal}$ has been calculated for values of thickness of 2 mm and 6 mm. It is shown that the values of the reduction factor $M_{u,T}/M_{u,normal}$ plots closely to the values of $M_{DSM,T}/M_{DSM,normal}$ within an acceptable range.

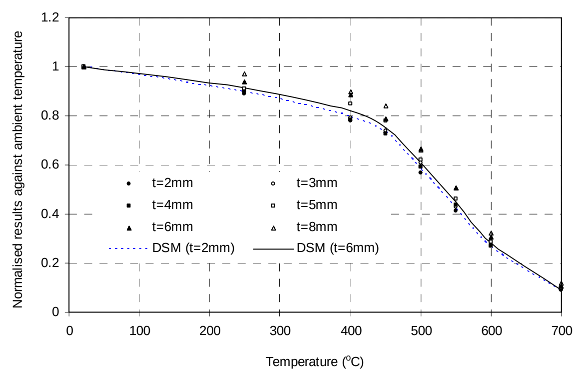
$M_{DSM,T}/M_{DSM,normal}$ is therefore appropriate for a design recommendation for the reduction in strength of a cold-formed steel channel-section undergoing premature web buckling at elevated temperatures.




(a) Bolt-group having $a_B$ = 200 mm




(b) Bolt-group having $a_B$ = 500 mm




(c) Bolt-group having $a_B$ = 1000 mm

Figure 8. Comparison of variation of moment capacity with direct strength method

## Concluding remarks

A simple design recommendation has been proposed that will allow premature buckling to be taken into account at elevated temperatures, by applying a reduction factor to the moment capacity determined at ambient temperature. This reduction factor is based on the moment capacity of the section, calculated using the direct strength method.

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**Appendix. - Notation**

| | |
|---|---|
| $a_B$ | length of bolt-group |
| $b_B$ | breadth of bolt-group |
| D | depth of web of channel-section |
| $E_{normal}$ | Young's modulus at normal room temperature |
| $E_T$ | Young's modulus at temperature T°C |
| $f_y$ | yield stress |
| $f_{y,normal}$ | yield stress at normal room temperature |
| $f_{y,T}$ | yield stress at temperature T°C |
| $f_u$ | ultimate stress |
| $f_{u,normal}$ | ultimate stress at normal room temperature |
| $f_{u,T}$ | ultimate stress at temperature T°C |
| $M_{DSM,normal}$ | moment capacity calculated using direct strength method at normal room temperature |
| $M_{DSM,T}$ | moment capacity calculated using direct strength method at temperature T°C |
| $M_u$ | ultimate moment capacity |
| $M_{u,normal}$ | ultimate moment capacity at normal room temperature |
| $M_{u,T}$ | ultimate moment capacity at temperature T°C |
| t | thickness of channel-section or plate |