A fire resistance design method for thin-walled steel studs in wall panel constructions exposed to parametric fires

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A B S T R A C T

This paper investigates the applicability of a simple fire resistance design method for axially loaded thin-walled steel studs in wall panel assemblies when exposed to parametric fires from one side. The simple method includes calculations of cross-section temperatures and ultimate load carrying capacities at elevated temperatures. The simplified calculation method for heat transfer in the cross-section is based on dividing the cross-section into a number of segments. The thermal properties of these layers are based on weighted averages of the thermal properties of the components contained within. The structural capacity calculation method is based on the Direct Strength Method. Results from the design method are compared with the results from Finite Element simulations for heat transfer and structural analysis (236 models). The calculation results are in good agreement with the simulation results and the proposed method may be used in performance-based fire engineering design of such construction.

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1. Introduction

Fire resistance is one of the most fundamental requirements of building safety. Two different approaches may be employed to demonstrate structural fire safety: prescriptive approach based on specifying fire protection thickness to achieve a standard fire resistance rating [1] and performance-based method in which the behaviour of the structure in fire is explicitly assessed [2]. In the performance-based approach, realistic fire conditions may be considered by using the so-called parametric fire curves [3]. This paper considers performance-based fire safety design of thin-walled steel studs.

Cold-formed thin-walled steel structures are one of the common load bearing members in building construction as they are easy to construct with a short construction time. Usually they are part of wall panel assemblies (Fig. 1), which consist of cold-formed thin-walled steel sections (commonly lipped channel), thermal insulation and gypsum plasterboard. Gypsum boards act as fire protection and they do not carry any axial load.

Cold-formed thin-walled steel members have much more complex behaviour than hot-rolled members in fire. Not only are they prone to various types of buckling (local, distortional and global), they also have non-uniform temperature distributions in the cross-section due to fire exposure from one side (Fig. 2) and the presence of the gypsum plasterboards and interior insulation. Whilst fire resistant design methods for hot-rolled members [1,4], including allowing for natural (parametric) fire conditions, are well established, the development for design methods for thin-walled members in fire is still at a relatively early stage.

Evaluating fire resistance of a structural member includes obtaining temperature distributions in the cross-section first and then calculating the structural load-bearing capacity at elevated temperatures. For structural performance assessment of thin-walled studs in wall panels under compression, the authors [5–7] have developed methods for calculating their ultimate load carrying capacity under global, local and distortional buckling with non-uniform temperature distribution in the cross-section. The developed methods are based on the direct strength method (DSM) [8] which has been adopted by AISI [9] for ambient temperature applications. In DSM, the critical elastic buckling loads and the cross-sectional plastic loads are evaluated separately. They are then used in a similar way to obtain the compressive resistance of the member as in hot-rolled column design by using buckling curves. The authors’ extensions of DSM to elevated temperature calculation will be summarised in Section 5.2.

For structural load carrying capacity calculations, the non-uniform temperature distribution in the thin-walled cross-section may be represented by linear distribution between the two flanges, therefore, it is only necessary to obtain the temperatures of the two flanges in heat transfer analysis. This conclusion was based on the conclusion of Feng et al. [10]. To obtain these two temperature values, the authors recently [11] developed a simple...
approach which is based on simple finite difference solution of one dimensional heat transfer in the panel thickness direction and uses the weighted average of thermal resistances in the panel width direction. This temperature calculation method has been demonstrated to be accurate for application under different fire conditions and this method will be summarised in Section 5.1.

Combining the proposed temperature calculation method and structural load carrying capacity method allows the fire resistance to be easily evaluated in design. The applicability of the combined thermal–structural performance method has been assessed under the standard fire condition (heating stage) and the results are very accurate [11]. This paper investigates applicability of this method under parametric fire conditions which are now becoming more widely used in practical performance-based fire engineering design. The effects of cooling will also be included.

2. Assumed parametric fire temperature–time curves

EN 1991-1-2 [3] gives an equation for evaluating the parametric fire curves which depend on the thermal properties of the fire enclosure, the opening factors (related to the area of openings relative to and total area of the enclosure) and the fire load. For the so-called standard fire enclosure with a thermal property value of $\sqrt{\rho c L} = 1160 \text{ J/m}^2 \text{ s}^{1/2} \text{K}$ and an opening factor of 0.04 m$^{1/2}$, the fire temperature–time relationship for the heating stage is almost identical to the standard fire temperature–time equation.

Different thermal properties and opening factors will result in faster or slower rates of increase in the fire temperature. In this study, three representative fire temperature–time relationships will be used. These are the standard curve (medium fire, $\theta_e = \theta_a + 345 \log_{10}(8t+1)$), fast fire ($\theta_e = \theta_a + 445 \log_{10}(8t+1)$) and slow fire ($\theta_e = \theta_a + 245 \log_{10}(8t+1)$). In addition, a cooling phase is associated with each fire curve, starting at 20 min, 40 min and 60 min respectively for the fast, medium and slow fires. The fire temperature–time relationships during the cooling phase are according to EN 1991-1-2. The three fire temperature–time curves are described by the equations below:

- Fast fire
  
  For heating phase: $\theta_e = \theta_a + 445 \log_{10}(8t+1)$, $t_{\text{max}} = 20 \text{ min}$

  $t_{\text{max}} = 20 \text{ min}$

- Medium fire
  
  For heating phase: $\theta_e = \theta_a + 345 \log_{10}(8t+1)$, $t_{\text{max}} = 40 \text{ min}$

  $t_{\text{max}} = 40 \text{ min}$

- Slow fire
  
  For heating phase: $\theta_e = \theta_a + 245 \log_{10}(8t+1)$, $t_{\text{max}} = 60 \text{ min}$

  $t_{\text{max}} = 60 \text{ min}$

In the above equations: $\theta_e$ is the fire temperature; $\theta_a$ is the ambient or room temperature; $\theta_{\text{max}}$ is fire temperature at the maximum heating time and $t_{\text{max}}$ is the maximum heating time.

Fig. 3 shows the three representative fire temperature–time curves assumed in this study. These three fire curves produce a range of desired different temperature distributions in the cross-section.

3. Methodology

Fig. 4 shows the procedure used in validating the proposed method for calculating the fire resistance of thin-walled steel studs.
under parametric fire conditions. The calculation results were compared with the authors’ Finite Element simulation results using Abaqus Unified FEA software [12].

4. Finite element numerical study

In order to validate the proposed method (Section 5), a series of numerical studies have been performed by using Abaqus Unified FEA [12] software. The simulation models have been validated by the authors [5–7,11]. To reduce the computational time, the simulation was carried out in two phases:

- **Phase one**: 2D heat transfer analysis of the wall panel including the steel studs, the internal insulation and the gypsum boards. It was assumed that the temperature distribution along the height of the panel was uniform. Fig. 5 shows a sketch of the heat transfer model.
- **Phase two**: 3D structural analysis of the steel stud at elevated temperatures using the temperature profile from the heat transfer analysis.

In total 18 heat transfer and 236 structural analysis FE models were made. This allows the effect of different lipped channel sections, and three different fire temperature–time curves (Fig. 3) to be investigated. Table 1 summarises the numerical study cases.

**4.1. Heat transfer FE model**

Fig. 5 shows a typical heat transfer model: consisting of a steel stud (section size 100 × 56 × 15 × 2), thermal insulation (ISO-WOOL) and one layer of 12.5 mm thick gypsum plaster boards on both sides. The thermal properties of the steel stud are based on EN 1993-1-2 [13], those of the gypsum plaster board based on Rahmanian and Wang [14] and those of ISOWOOL (thermal insulation) based on Salhab and Wang [15].

The thermal boundary conditions (Fig. 6) are: initial (room) temperature = 20°C, resultant emmissivity = 0.3 on the exposed side and 0.8 on the unexposed side, convective heat transfer

![Diagram](image)

**Fig. 4.** Procedure of validation.

![Diagram](image)

**Fig. 5.** Typical wall panel with one layer of gypsum boards on both sides for heat transfer analysis.

**Table 1**

<table>
<thead>
<tr>
<th>Lipped channel section</th>
<th>Fire curve</th>
<th>Phase one (heat transfer)</th>
<th>Phase two (structural analysis models)</th>
</tr>
</thead>
<tbody>
<tr>
<td>75 × 50 × 15 × 2.5</td>
<td>Fast</td>
<td>114</td>
<td>Approximately every 10 min</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>128</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slow</td>
<td>138</td>
<td></td>
</tr>
<tr>
<td>100 × 50 × 15 × 2.5</td>
<td>Fast</td>
<td>114</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>128</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slow</td>
<td>138</td>
<td></td>
</tr>
<tr>
<td>100 × 50 × 5 × 1.8</td>
<td>Fast</td>
<td>114</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>128</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slow</td>
<td>138</td>
<td></td>
</tr>
<tr>
<td>100 × 50 × 15 × 0.8</td>
<td>Fast</td>
<td>114</td>
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<tr>
<td></td>
<td>Medium</td>
<td>128</td>
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<tr>
<td></td>
<td>Slow</td>
<td>138</td>
<td></td>
</tr>
<tr>
<td>100 × 75 × 15 × 0.8</td>
<td>Fast</td>
<td>114</td>
<td></td>
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<td></td>
<td>Medium</td>
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<td></td>
<td>Slow</td>
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<td>100 × 56 × 15 × 2</td>
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<td>114</td>
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<td></td>
<td>Medium</td>
<td>128</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slow</td>
<td>138</td>
<td></td>
</tr>
</tbody>
</table>

* Web depth × flange width × lip length × thickness.
coefficient on the fire exposed side = 25 W/m² K and on the unexposed side = 10 W/m² K. The element type is DC2D4 (4-node linear heat transfer quadrilateral) [11]. The temperature distribution in the steel studs from the heat transfer FE models were used in the structural analysis model using Abaqus.

4.2. Structural analysis FE model

The structural analysis is based on the validated Abaqus model [5–7] by the authors. The objective of this study is to compare the FE results with calculation results using the proposed method. Since the proposed method gives the load carrying capacity of the structure at elevated temperatures, the FE analysis was conducted in this way. To do so, the structure temperatures at any time were applied and then the mechanical load was applied until the steel stud reaching its ultimate load carrying capacity. The thermal expansion coefficient for steel was assumed to be 0.000014/°C. The mechanical properties at elevated temperature were based on EN 1993-1-2 [13] with a steel yield stress and elastic modulus 350 and 205,000 N/mm² respectively at ambient temperature.

For the FE structural analysis, the non-uniform structural temperatures were taken from the output of the FE heat transfer analysis (Section 4.1). Fig. 7 shows the mechanical boundary and loading conditions: the column was simply supported (preventing lateral displacement, allowing longitudinal displacement at each node except for the centre of one end) at the top and bottom of the column (Fig. 7b); the axial loads were applied on each node at the two ends of the column (Fig. 7b); it was restrained at 300 mm spacing in the lateral (flange) direction to simulate the restraining effect of the gypsum boards (Fig. 7c). It should be noted that in reality, the gypsum boards may lose their restraining effects on the steel studs in fire. However, since the objective of this study is to assess the proposed simple method, this effect was not considered and the assessment was considered valid as long as the same restraining conditions were applied in both the finite element model and the proposed method. The length of the column is 3000 mm. Abaqus shell element type S4 was used and the global mesh size was 2.5 mm. The FE structural analysis model incorporated initial imperfection. The initial imperfection mode corresponded to the expected final failure mode of the column. For distortional buckling, the maximum initial imperfection was equal to half of the section thickness; for pure local and pure global buckling, the maximum initial imperfection was equal to the thickness; for interactive local/global buckling, the maximum initial imperfection for the local buckling mode was equal to the thickness and the maximum imperfection for the global buckling mode was equal to H/500 (where H is the column height) respectively.

Fig. 6. Thermal boundary conditions.

Fig. 7. Mechanical boundary conditions. (a) column boundary, (b) loading condition and (c) restraints.
5. Proposed design method

5.1. Thermal analysis

In the research study by Feng et al. [10], it was found that for steel studs in wall panels exposed to fire attack from one side, the temperature distribution in the steel stud may be assumed to be linear between the hot and cold sides, with the flanges having uniform temperature distributions. This simplifies the temperature field in the panel structure considerably and lends it to the development of a simple method of heat transfer analysis. Recently the authors [11] have developed such a simple approach. In this approach, the wall panel is represented by a segment of width $W_e$ (Fig. 8), outside of which the panel temperature distribution is not affected by the steel stud. This segment is then divided into 13 layers through the panel thickness, as shown in Fig. 9. Heat transfer is assumed to be one dimensional in the thickness direction of the panel, with each layer having its own thermal conductivity and thermal capacitance (density times specific heat). The thermal properties of each layer are obtained from the weighted averages of the all materials contained in the layer within the segment. For calculating the contributions of different components within each layer to the overall thermal conductivity, this research applied the additive rule of thermal resistance (thickness/thermal conductivity) in the direction of heat conduction (conduits in series). In the direction perpendicular to heat conduction (parallel conductors), the additive rule of thermal conductivity (ratio of component thickness to overall layer thickness times component thermal conductivity) was used. The total thermal capacitance (density times specific heat) of the layer was assumed to be the sum of all the components based on their proportion to the total mass. Based on these assumptions, a 1D finite difference program has been written to calculate the temperatures of all the layers. This method is much quicker than using 2D finite element heat transfer analysis, but more importantly, it can be easily adopted in design.

This procedure has been implemented for panels exposed to the standard fire from one side and comparison against the results of an extensive set of finite element heat transfer simulations comprehensively validated this procedure [11]. This paper (Section 6) provides further validation of this method for panels exposed to parametric fires with cooling stage. The width of the segment (Fig. 8) for calculating the weighted average of thermal properties is

$$W_e = t_i + b_f + b_i = 45 + 0.85 b_f \text{(in mm, } t_i = b_i)$$

5.2. Extended direct strength method for fire applications

Direct strength method [8] is a recently adopted method [9] for calculating the load carrying capacity of thin-walled columns. It is based on regression equations using the elastic critical load and the plastic resistance of the column. The elastic buckling load for local, distortional and global buckling can be obtained by using a numerical procedure such as CUFSM which is a Finite Strip Method program [16]. The plastic resistance of the cross-section (squash load) is simply the gross sectional area multiplied by the yield stress.

Recently the authors [5–7] have applied this method to fire conditions, for both uniform and non-uniform temperature distribution in the cross-section.

For calculating the elastic buckling loads using CUFSM, the different strips of the cross-section are assigned different Young’s modulus values according to their temperatures. Without a non-uniform temperature distribution in the cross-section, the effects of thermal bowing and shift of the centre of resistance generate bending moments in the cross-section. Therefore, the original column under pure axial load becomes a beam-column with combined axial load and bending moment. The squash load of the column used in DSM is the axial load value that causes the cross-section with non-uniform temperature to reach the full plastic limit of the cross-section under combined axial load and bending. This is termed “effective” squash load in this study and is obtained using the following procedure [5]:

1. Determine the plastic axial load–bending moment interaction curve of the cross-section. To do so, locate a neutral axis in the cross-section, which divides the cross-section into a tensile and a compressive part. Assume the cross-section has reached complete yield everywhere. The resulting stress distribution gives one value of axial load and one value of bending moment which is taken about the centre of resistance of the cross-section with non-uniform temperature distribution. Moving the neutral axis from one extreme side of the cross-section to the other side gives the continuous plastic axial load–bending moment interaction curve of the cross-section.

2. Since the top and bottom of the cross-section are fixed in position, there is no thermal bowing displacement so only a shift of the centre of resistance exists. In the middle of the column, both thermal bowing and the shift of the centre of resistance exist and they are in opposite directions. As a result, the bending moments at the ends and in the middle of the column are different. The effective squash load of the column is the one that gives the least value based on these two bending moments.
For application at elevated temperatures, the ambient temperature buckling curves had to be modified to take into consideration the difference in effects of initial imperfections at ambient and elevated temperatures. The modified buckling curves are:

- For global buckling [5]:
  \[
  \lambda_c = \sqrt{\frac{P_y}{P_{cre}}}
  \]
  for \( \lambda_c \leq 1.5 \), \( P_{ne} = (0.495 \lambda_c^2)P_y \) \quad (5a)
  for \( \lambda_c > 1.5 \), \( P_{ne} = \left(\frac{0.462}{\lambda_c^2}\right)P_y \) \quad (5c)

- For local buckling (including local–global interaction) [6]:
  \[
  \lambda_l = \sqrt{\frac{P_{ne}}{P_{cri}}}
  \]
  for \( \lambda_l \leq 0.776 \), \( P_{nl} = P_{ne} \) (6a)
  for \( \lambda_l > 0.776 \), \( P_{nl} = \left(1 - 0.22\left(\frac{P_{crit}}{P_{ne}}\right)^{0.75}\right)\left(\frac{P_{crit}}{P_{ne}}\right)^{0.75}P_{ne} \) (6c)

- For distortional buckling [7]:
  \[
  \lambda_d = \sqrt{\frac{P_y}{P_{crd}}}
  \]
  for \( \lambda_d \leq 0.561 \), \( P_{nd} = P_y \) (7b)
  for \( \lambda_d > 0.561 \), \( P_{nd} = 0.65\left(1 - 0.14\left(\frac{P_{crit}}{P_y}\right)^{0.7}\right)\left(\frac{P_{crit}}{P_y}\right)^{0.7}P_y \) (7c)

Fig. 10 compares the authors’ modified and the original DSM interaction curves.

The axial strength of the column is the minimum of \( P_{ne} \) (for global buckling), \( P_{nl} \) (for local buckling) and \( P_{nd} \) (for distortional buckling). In the above equations: \( P_y \) is the “effective” squash load of the cross-section (taking into consideration the effects of thermal bowing and shift in the centre of resistance as explained.

---

**Fig. 10.** Original and modified DSM buckling curves [5–7,9].

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**Fig. 11.** Comparison of average flange temperatures for LC100 × 56 × 15 × 2: (a) slow fire growth, (b) medium fire growth and (c) fast fire.
6. Evaluation of the proposed design method: comparison with Abaqus simulation results

6.1. Comparison of temperature results

Fig. 11 shows typical comparison of the average flange temperatures on the exposed and unexposed sides of the section between the proposed simple calculation method and Abaqus simulation results for section 100 × 56 × 15 × 2 with one layer of gypsum board on both sides. The comparison indicates excellent agreement, including the plateau stage. The simple method gives
slightly conservative results before the temperature reaches their maximum. Fig. 12 further shows that the agreement for temperature distributions in the cross-section at different times (30 and 60 min) is very good for all three fire curves.

6.2. Comparison of column strength

Section 5.2 has provided a description of how the effects of thermal bowing and shift of centre of resistance can be included in the calculations for the elastic buckling loads and the “effective” squash load. In order to do so, it is necessary to calculate the thermal bowing and the shift of centre of resistance. For the thermal bowing deflection, Fig. 13 shows a comparison between Abaqus FEA results and the analytical equation \((aL^2 \Delta T / 8d)\) [1]. The agreement is very good.

Fig. 14 shows the shift of the centre of resistance of the cross-section in time. At ambient temperature, the centre of resistance is the same as the centre of the cross-section because the mechanical properties are distributed uniformly in the cross-section. However, when the mechanical properties are non-uniform in the cross-section due to non-uniform high temperatures, the centre of resistance is different from the geometrical centre of the cross-section. With slower heating, the temperature gradient in the cross-section is smaller, hence the shift of the centre of resistance is smaller. However, in all cases, the maximum shift of the centre of resistance is quite significant. This shift gradually returns to the minimum when the cross-section cools down and the temperature in the cross-section return to ambient temperature. To demonstrate the importance of including the effects of the thermal bowing and shift of the centre of resistance, Fig. 15 compares the squash loads with and without considering these effects.

Appendix demonstrates how the DSM method is applied, at given flange temperatures, through an example. Fig. 16 compares the column strengths at different fire exposure times. The agreement is

<table>
<thead>
<tr>
<th>Lipped channel section</th>
<th>Ambient temperature</th>
<th>Elevated temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Abaqus (P_{FE})</td>
<td>DSM (P_u)</td>
</tr>
<tr>
<td>(75 \times 50 \times 15 \times 2.5)</td>
<td>89,840.20</td>
<td>87,711.50</td>
</tr>
<tr>
<td>(100 \times 50 \times 5 \times 1.8)</td>
<td>71,864.20</td>
<td>80,925.73</td>
</tr>
<tr>
<td>(100 \times 50 \times 15 \times 0.8)</td>
<td>27,210.40</td>
<td>23,400.84</td>
</tr>
<tr>
<td>(100 \times 50 \times 15 \times 2.5)</td>
<td>144,323.00</td>
<td>130,114.40</td>
</tr>
<tr>
<td>(100 \times 56 \times 15 \times 2)</td>
<td>115,000.00</td>
<td>112,873.66</td>
</tr>
<tr>
<td>(100 \times 75 \times 15 \times 0.8)</td>
<td>24,827.70</td>
<td>27,458.33</td>
</tr>
</tbody>
</table>

Note: \(P_{u_{min}} \text{[N]}\) = minimum column capacity.
generally very good. A comparison between using the Abaqus and proposed temperature calculation results, given in Table 2, indicate that although there is some difference in the temperature calculation results, the influence of these temperature differences is small on the column strengths.

The proposed method gives slightly higher minimum column strength in most cases. This is because the proposed modified Direct Strength Method equations (Eqs. (7a–c)) give slightly higher values than the FE simulation results when the load ratio is small. Here the load ratio is the ratio of the strength in values than the FE simulation results when the load ratio is small.

Direct Strength Method equations (Eqs. (7a–c)) give slightly higher values than the FE simulation results when the load ratio is small. Here the load ratio is the ratio of the strength in values than the FE simulation results when the load ratio is small.

Using the proposed simplified calculation method is slightly unconservative than the Abaqus simulation results when the load ratio is low. However, the overall agreement between the two sets of results is very good. In summary, despite the simplicity of the proposed thermal and strength calculation methods, both the temperature and column strength values are calculated with very good accuracy when compared with the Abaqus FEA simulations.

7. Conclusion

This paper has presented the results of an extensive set of FE simulations and a semi-analytical method containing models for thermal and structural performance of wall panel assemblies consisting of thin-walled steel studs, thermal insulation and gypsum/plaster boards. The panels were exposed to parametric fires, including the cooling stage, from one side. The proposed semi-analytical method includes using a simple heat balance procedure to calculate the approximate temperature distribution in the steel stud, the finite strip program CUFSM to calculate the elastic critical loads at elevated temperatures, and a procedure to calculate the effective squash load of the cross-section at elevated temperatures including the effects of thermal bowing and the shift in the centre of resistance of the cross-section. These semi-analytical methods have previously been applied to thin-walled steel studs in wall panels exposed to the standard ambient temperature. To demonstrate this, Fig. 17 plots the ratio of the calculated column strength using the proposed method to that using Abaqus simulation against load ratio. The solid marks (for the minimum strength) follow the general trend for global buckling, which is that the proposed simplified calculation method is slightly unconservative than the Abaqus simulation results when the load ratio is low. However, the overall agreement between the two sets of results is very good. In summary, despite the simplicity of the proposed thermal and strength calculation methods, both the temperature and column strength values are calculated with very good accuracy when compared with the Abaqus FEA simulations.

Appendix A. A worked example as illustration of the calculation procedure

Requirement: Calculate the load carrying capacity of a panel at 50 min of the medium parametric fire exposure time.

Basic input data: Steel section dimensions—lipped channel 75 × 50 × 15 × 2.5; Interior insulation—ISOWOOL; Gypsum—One layer of 12.5 mm thick Fireline British gypsum on each side; Panel height—3 m.

First step: Calculate the steel temperatures in the flanges on the fire exposed and unexposed sides. The equivalent panel width for calculating the average steel temperature is

\[ W_e = 45 + 0.85(50) = 87.5 \text{ mm} \]

Using the proposed method which should be implemented and solved numerically by computer program (http://j.mp/TRWCW1), the temperature–time results in Fig. A1 can be obtained. From this figure, the steel flange temperatures at 50 min on the exposed and unexposed sides are 454.84°C and 264.91°C respectively.

Second step: Determination of the effective squash load.

Using method described briefly in Section 5.2 and in detail in [5], the centre of plastic resistance is 32.93 mm from the lower temperature flange side (37.5 mm at ambient temperature). Therefore, the shift of centre of resistance is 37.5−32.93 = 4.57 mm.

Using equation in Fig. 12 gives a thermal bowing [5] value of 39.88 mm. Therefore, the eccentricities at the top and bottom and at the centre of the column are 4.57 mm and |4.57−39.88| = 35.31 mm respectively. The column axial load−bending moment interaction curve is obtained based on Section 5.2 (http://j.mp/CALESL), as shown in Fig. A2. From the two eccentricities above, the effective squash loads are 113.14 kN (top and bottom) and 67.81 kN (middle) respectively. Therefore, the effective squash load is \( \min(113.14, 67.81) = 67.81 \text{ kN} \).

Using the CUFSM program (http://bit.ly/CUFSM), with input of the elevated temperature Young’s modulus values of the steel
section, the elastic critical loads for global, distortional and local buckling are:

\[ P_{crg} = 72.96 \text{ kN} \]
\[ P_{crd} = 410.22 \text{ kN} \]
\[ P_{crl} = 345.81 \text{ kN} \]

Using Eqs. (5)–(7):

\[ \lambda_c = 0.9640; \quad P_{nl} = 35.27 \text{ kN} \]
\[ \lambda_l = 0.2932; \quad P_{nl} = 35.27 \text{ kN} \]
\[ \lambda_d = 0.4428; \quad P_{nd} = 67.81 \text{ kN} \]
\[ P_n = \min(35.27, 35.27, 67.81) = 35.27 \text{ kN} \]

For comparison, the finite element solution is 31.19 kN.

References