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Direct Strength Method for Cold-Formed Steel Beams with Non-Uniform Temperatures

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Abstract

This paper presents the results of finite element simulations leading to the development of a design method using the Direct Strength Method (DSM) for transversely loaded thin-walled steel beams prone to local and distortional buckling failures at elevated temperatures.

The systematic and extensive numerical parametric study covers different dimensions of thin-walled steel sections, different temperature distributions in the steel cross-section, different steel grades, and different load ratios. The main findings are that DSM is a suitable method for thin-walled steel members with non-uniform elevated temperature distributions in the cross-section. Based on using the plastic moment capacity of the cross-section at elevated temperatures, the existing AISI (2016) DSM equations are sufficiently accurate for local buckling. For distortional buckling, a set of new DSM equations are proposed.

The proposed DSM equations are then used to derive partial safety factors for structural resistance for conditional probabilities of structural failure of 0.1, 0.01 and 0.001 after flashover.

Keywords chosen from ICE Publishing list

Beams and girders, Fire engineering, Risk and Probability analysis

List of notations

b	cross-section flange width
d	cross-section lip depth
E	Young's modulus of steel
E_{20}	Young's modulus of steel at ambient temperature
E_T	Young's modulus of steel at temperature, T
f_y	yield stress of steel
$f_{y,20}$	yield stress at ambient temperature
$f_{y,T}$	yield stress at temperature, T
h	cross-section depth
M_{crd}	critical elastic distortional buckling load
M_{crl}	critical elastic local buckling load
M_{FEM}	moment capacity from finite element modelling
M_{nd}	bending resistance for distortional buckling, based on DSM
M_{ne}	bending resistance for global buckling, based on DSM
M_{nl}	bending resistance for local buckling, based on DSM
M_p	plastic moment capacity of beam
M_y	first yield moment of beam
t	cross-section thickness
U_x	displacement in x direction
U_y	displacement in y direction
U_z	displacement in z direction
ε	radiative emissivity
θ_z	twist about z direction
λ_d	distortional buckling slenderness
λ_l	local buckling slenderness
μ	Poisson's ratio

1 Introduction

The use of CFS (cold-formed steel), thin-walled construction has grown owing to their numerous advantages including high strength to weight ratio, flexibility of manufacture, fast and easy construction as well as low handling and transportation costs (Hancock *et al*., 2001).

As structural members, in addition to their traditional use as purlins and in other secondary structures such as panels and floor decking, there is now widespread use of thin-walled steel sections as main loadbearing structural members such as in lightweight portal frames, studs, columns and beams.

Fire safety is critical to thin-walled steel building structures. In particular, thin-walled steel sections are usually part of a panel system exposed to fire from one side. Therefore, while the inclusion of fire protection materials and interior insulations improve their thermal and structural performances, they result in non-uniform temperature distribution in the cross-section of thin-walled structures. Care must be taken in understanding the structural behaviour as a result of non-uniform mechanical property distribution in the cross section of thin-walled steel beams. Non-uniform temperature distributions not only introduce thermal bowing (Wang and Davies, 2000), they also result in shift of the centre of resistance.

Whilst there have been a large number of research studies to investigate the fundamental behaviour of thin-walled steel structures in fire and how to improve their fire resistance (e.g. Feng *et al*. (2003); Feng and Wang (2005); Kankanamge (2010); Kankanamge and Mahendran (2012); Baleshan (2012); Baleshan and Mahendran (2016); Laím *et al*. (2014); Cheng *et al*. (2015); Landesmann and Camotim (2016); Jatheeshan and Mahendran (2016); Wang *et al.* (2020)), design calculation methods for fire resistance of thin-walled steel structures are still rudimentary.

The effective width method (EWM) and the direct strength method (DSM) are two candidates for consideration in the development of fire resistance design methods for thin-walled steel structures, as for ambient temperature design. As noted by (Schafer, 2002a), EWM has a number of shortcomings: it calculates the effective widths of the different elements of a thin-walled steel section element-by-element thus ignoring element interactions; also it becomes cumbersome for members with longitudinal stiffeners. To overcome these shortcomings, the direct strength method (DSM) has been developed (Hancock *et al*. (1994); Schafer and Peköz (1998a); Schafer (2002b)). DSM combines the elastic buckling load of the member and the yield resistance of the cross-section to calculate the member resistance (Schafer and Peköz, 1998a). Simple computer programs, such as Cornell University Finite Strip Method (CUFSM) by Schafer and Ádány (2006) can be used to calculate the elastic buckling load in which inter-element interactions are considered. The direct strength method has now been incorporated into a number of design codes such as AISI (2016) and Australian code AS/NZS 4600 (SA, 2018). For fire resistance design of thin-walled structures with non-uniform temperature distribution, the advantages of DSM over EWM are more profound: there is no need in DSM to tackle the difficult challenge of dealing with non-uniform mechanical properties in calculating effective widths of different plate elements. In DSM, it is relatively straightforward to incorporate non-uniform temperature distribution in the cross-section when calculating elastic buckling load and plastic cross-section property.

Because of these advantages, DSM is now mainly pursued by researchers. Results of studies by Heva and Mahendran (2008), Ranawaka and Mahendran (2009), Gunalan *et al*. (2015) show that except for a few cases, DSM is applicable to uniformly heated and axially compressed thin-walled steel sections. Landesmann *et al*. (2019) have demonstrated that DSM is reasonably accurate for axially compressed members undergoing distortional buckling failure with high slenderness. Shahbazian and Wang (2011a), Shahbazian and Wang (2011b), Shahbazian and Wang (2012) developed DSM equations for calculating compression resistance of thin-walled steel studs at high temperatures, allowing for some bending moment.

The context of their research is the situation when a thin-walled steel stud under compression is exposed to fire from one side. Therefore, additional bending moments are generated in the member due to *P*-*δ* (caused by thermal bowing) effect and the effect of shift of centre of resistance. The have proposed new DSM equations for different failure modes (local buckling, distortional buckling, and global buckling) under different heating conditions (standard fire exposure, parametric fire exposure).

In contrast, research studies to assess applicability of DSM to flexural members at elevated temperatures are limited. Studies by Kankanamge (2010) and Landesmann and Camotim (2016) are limited to global and distortional buckling failure modes under uniform bending at uniform elevated temperatures. Even so, Landesmann and Camotim (2016) reported that DSM could not predict the distortional buckling strength of uniformly heated thin-walled steel sections under pure bending.

There is a clear need to develop fire resistance design methods for thin-walled steel members under bending. DSM has many advantages compared to EWM. Therefore, the aim of the present study is to investigate suitability of DSM for fire resistance design of thin-walled steel members under bending with non-uniform heating in the cross-section.

This research is carried out by a systematic and comprehensive evaluation of numerical parametric simulation results using the general finite element software ABAQUS (Abaqus, 2014). Transient state structural-thermal analysis is performed so that realistic temperature profiles are used in simulations of structural response. Since ABAQUS is used for both heat transfer modelling and structural analysis, it is necessary to demonstrate its validity.

2 Numerical Heat Transfer Analysis

This research considers the following three types of panel construction for validation of heat transfer analysis:

1. Fire protection boards on both sides without internal insulation;
2. Fire protection boards on both sides with external insulation;
3. Fire protection boards on both sides with internal insulation.

To ensure accuracy of heat transfer modelling, relevant fire tests by others are simulated.

2.1 Steel sections with fire protection on both sides without interior insulation

Baleshan (2012) and Jatheeshan and Mahendran (2015) carried out fire tests on thin-walled steel sections with open channel and hollow flange channel sections respectively. The joist arrangements are shown in Figure 1. The thickness of each layer of gypsum plasterboard and plywood is 16 mm and 19 mm respectively. The section dimensions are: web depth 180 mm, flange width 40 mm, lip depth 15 mm, thickness 1.15 mm for the tests of Baleshan (2012), and web depth 200 mm, flange width 45 mm, lip depth 15 mm, thickness 1.6 mm for the tests of Jatheeshan and Mahendran (2015).

The present ABAQUS heat transfer model has the following main features:

1. *Emissivity of 0.9 on both the ambient and fire sides as proposed by Keerthan and Mahendran (2012);*
2. *Convective heat coefficients 25 W/m^2K and 10 W/m^2K for the fire and ambient sides respectively by Keerthan and Mahendran (2012);*
3. *Cavity radiation (ε=0.6) for voids between the steel sections;*
4. *Standard fire exposure;*
5. *At least two elements of type DC2D4 (4-node linear heat transfer quadrilateral element) each in the steel thickness direction;*
6. *6 elements in the thickness direction and 20 mm mesh in the longitudinal direction for the plasterboard and plywood.*
7. *TIE constraint to account for heat transfer by conduction between components.*

8. *Stefan-Boltzmann constant of* 5.67×10^{-8} W/m^2K^4

For gypsum plasterboard, the proposals of Keerthan and Mahendran (2012), Rahmanian and Wang (2012) and Jatheeshan and Mahendran (2016) are adopted for its specific heat capacity, density and thermal conductivity respectively, as suggested by Alabi-Bello and Wang (2018) and shown in Figure 2, Figure 3 and Figure 4.

The thermal properties of steel (density, specific heat capacity and thermal conductivity) are according to EN 1993-1-2 (CEN, 2005).

For plywood, the thermal properties proposed by Jatheeshan and Mahendran (2016) are used. The density and specific heat capacity values are 500 kg/m^3 and 1.5kJ/kg°C respectively. Figure 5 shows the temperature dependent thermal conductivity of the material.

The results in Figure 6 indicate acceptable agreement between test and numerical simulation results, in particular in the important stage after evaporation of water from plasterboard when the exposed side steel temperatures are high. The differences between numerical modelling and test results may be attributed to uncertainties in thermal properties of the fire protection materials (plasterboard), as encountered by a number of previous researchers such as Mehaffy *et al*. (1994), Sultan (1996), Thomas (1997), Rahmanian and Wang (2012) and Keerthan and Mahendran (2012).

2.2 Steel sections with fire protection on both sides with internal or external thermal insulation

Baleshan (2012) and Jatheeshan and Mahendran (2015) carried out fire tests on thin-walled sections with external fire protection on both sides and external or internal insulation, as shown in Figure 7(a) and Figure 7(b).

The same simulation parameters and material thermal properties for plasterboard as in the previous section were used to simulate these tests.

For the rockwool insulation, the thermal properties described in Keerthan and Mahendran (2013) are used. The density and specific heat capacity values are 100 kg/m^3 and 0.84kJ/kg°C respectively while Figure 8 shows its temperature dependent thermal conductivity.

Figure 9 presents comparisons of results between the simulations and the tests of (Baleshan, 2012) and (Jatheeshan and Mahendran, 2015). Again, the simulation results can be considered acceptable.

Overall, the numerical simulation model can be considered acceptable for heat transfer analysis.

3 Validation of numerical models for structural behaviour

Prior to carrying out nonlinear analysis for structural behaviour at elevated temperatures, it is necessary to determine the failure loads of the test CFS thin-walled flexural members at ambient temperature.

3.1 Comparison against ambient temperature tests for bending

The tests of Baleshan (2012) was selected for comparison. Figure 10 shows a typical finite element configuration for one of the test beams, and the following describes relevant test conditions and simulation features.

Finite element mesh size

While retaining 4 elements each in the lips, a 5 mm mesh size is used for the flanges, the web and in the longitudinal direction. Shell element S4R is used for all elements.

Boundary conditions

To simulate simple supports used in the tests, the nodes at both ends are restrained against movement in the lateral directions (U_x = U_y = 0) and rotation about the longitudinal direction

(θ_z =0). At one end, the mid-web node is additionally restrained against movement in the longitudinal direction (U_z = 0).

It was reported by Baleshan (2012) that the plasterboard/plywood provided sufficient lateral and torsional restraint of the steel sections. To simulate the restraining effect of plasterboards and plywood on the beam sections, the flange nodes are restrained against lateral displacement (U_x =0) and twist about the longitudinal direction (θ_z =0) at locations of 300 mm intervals along the top flange and 200 mm intervals along the bottom flange nodes respectively. These locations represent the screw points in the tests as reported by Baleshan (2012). The beam span was 2.4 m.

Material properties

The measured ambient temperature mechanical properties are used. Elastic-perfectly plastic stress-strain curve is adopted. For the tests reported by Baleshan (2012), the yield stress is 612 MPa and the Young's modulus and Poisson's ratio are 210260 MPa and 0.3 respectively.

Loading

Uniformly distributed load in the form of a total pressure load is applied on the compression flange of the beam, in accordance with the reported test loading condition.

Residual stresses

Schafer *et al*. (2010) concluded that the effects of residual stress may be offset by the increased yield stress in the corner regions (cold-work effects) of the section. Therefore, neither was considered in this research.

Initial imperfections

The magnitude of initial imperfection for the first local buckling mode is assumed to be 0.34t (where t is section thickness), which corresponds to the 50% CDF (cumulative distribution function) value as recommended by Schafer and Peköz (1998b). The buckling mode is determined via elastic buckling analysis in ABAQUS.

Figure 11 compares moment-deflection curves between the finite element modelling and those of the quoted researcher. The result in Figure 11 demonstrates good agreement.

3.2 Comparison against fire test results

Transient analysis is carried out as in the fire tests of Baleshan (2012). It involved using 2 static, general steps in ABAQUS:

i. Step1 –Mechanical load applied incrementally up to the target value; and

ii. Step 2- Temperatures incrementally applied to the member until failure.

The mechanical load is 40% (9kN) of the ambient temperature peak load. Figure 12 shows the configurations of the CFS sections.

To minimise the effects of any inaccuracy in heat transfer on modelling structural response, the reported test steel temperatures are used for validation of structural analysis. Figure 13 shows the recorded test temperature distributions for one of the channel sections.

Table 1 lists the elevated temperature mechanical properties (yield stress, *fy* and Young's modulus, E) as proposed by Kankanamge and Mahendran (2011) and thermal expansion according to Lie (1992).

Kankanamge and Mahendran (2011) fitted Ramberg-Osgood model (Ramberg and Osgood, 1943) equations to their measured stress-strain curves and these are shown in Figure 14 for tests of Baleshan (2012).

Figure 15 compares the simulation results with the test results of Baleshan (2012) and the simulation results of Baleshan and Mahendran (2016). The agreement is very good, especially the numerical modelling results are very close to the simulation results of Baleshan and Mahendran (2016).

Furthermore, Figure 16 confirms that the present simulation gives the failure mode as that of Baleshan (2012).

Overall, the simulation model can be considered validated for modelling flexural behaviour of CFS members at ambient temperature and in fire.

4 Numerical parametric study and assessment of fire resistance design method

4.1 Simplification of steel temperature distribution

Temperature distributions in the cross-sections of thin-walled steel sections exposed to fire from one side are non-uniform and non-linear, as shown in Figure 17 for the fire tests of the validation study in the previous section.

Inclusion of the exact non-uniform temperature distribution in subsequent structural-thermal analysis is time consuming due to the need to transfer the non-uniform temperature results to multiple lines of nodes along the flanges, the lips and the web. This section examines whether it would be possible to use simplified temperature distributions in the thin-walled steel cross-section.

This assessment is based on the numerical heat transfer analyses results of the same fire tests of Baleshan (2012) as in the validation study of the previous section.

In the simplified temperature distribution for CFS sections with interior insulation, it is assumed that the hot flange/lip have the same temperature (magnitude equal of that of the mid-flange), the cold flange/lip have the same temperature (that of the mid-flange) and the temperature distribution in the web is bilinear, intersecting at the position of interior insulation as shown in Figure 18(a). For CFS sections without insulation, the web temperature distribution is assumed to be linear as shown in Figure 18(b). In addition, uniform temperature distribution is assumed along the member length.

Figure 19 compares the simulation results between using the actual temperature distributions in ABAQUS modelling and those using the simplified temperature distributions for the same load ratio (80% of the ambient temperature load carrying capacity).

The two sets of results in each case are almost identical, confirming that it is acceptable to use the assumed simplified temperature distributions in Figure 18.

4.2 Numerical parametric study results and assessment of direct strength method

The parametric study investigates the effects of changing all design variables, including beam length, cross-section size and yield stress, to generate a comprehensive database for both buckling failure (local and distortional) modes. Table 2 summarises the parameters and their values. The steel grades of 275, 355 and 450 are nominal standard steel grades, and the steel yield stress value of 612 N/mm^2 was used in the tests of Baleshan (2012), with a Young's modulus (E) of 210 GPa and Poisson's ratio (μ) of 0.3. In addition to the parameters in Table 2, for simulations at elevated temperatures, the load ratio ranges from 0.3 to 0.8. The member lengths have been chosen to be sufficiently long in order to minimise the effect of shear due to transverse loading. Additionally, the selected sections in Table 2 were chosen so that the lowest critical buckling modes obtained from preliminary buckling analyses are the same as intended.

The non-uniform temperature distributions are for three possible configurations of thin-walled steel floor beams, as shown in Figure 20: no interior insulation in the cavity (Case 1), half insulated cavity (Case 2) and fully insulated cavity (Case 3). Case 3 is an addition to cases 1 and 2 of Baleshan (2012). The plasterboard and plywood are 16 mm thick while depth of interior insulation depends on section size. The steel section temperatures are generated by heat transfer modelling using the validated ABAQUS model, and then simplified, as explained in the previous section. The parametric study also includes ambient temperature results which will be used to compare the simulation results with existing ambient temperature DSM (AISI, 2016) equations for thin-walled beams.

For simulations at elevated temperatures, the elevated temperature mechanical properties (yield stress, f_y and Young's modulus, E) are as proposed by Kankanamge and Mahendran (2011). This model is adopted because of its ability to take into consideration different grades of steel. The coefficient of thermal expansion is according to Lie (1992). The Ramberg-Osgood model of Kankanamge and Mahendran (2011) is used to prepare the ABAQUS input stress-plastic strain data, with suitable modifications to allow for the initial linear proportion and strain hardening.

The magnitude of initial imperfection for the local buckling mode is assumed to be 0.34t (where t is the section thickness) and 0.94t for distortional buckling mode, as recommended by Schafer and Peköz (1998b). The buckling mode is determined via elastic buckling analysis in ABAQUS.

Transient state non-linear structural-thermal analysis is adopted to mimic the effects of fire on structures. In the analysis, the beam is incrementally loaded up to a target mechanical load value and then subsequently heated up (by increasing the beam temperatures) until failure.

4.2.1 Assessment of DSM for local buckling of beams at ambient temperature

Figure 21 shows typical ultimate failure mode at ambient temperature. The figure shows that the numerical simulation model failed in local buckling mode as intended.

Table 3 lists all the simulation results and Figure 22 presents all the simulation results for ambient temperature, plotted to show ratio of beam bending resistance/cross-section plastic moment – beam slenderness relationship. To calculate the beam slenderness, the beam critical elastic buckling load is determined using ABAQUS.

The results in Figure 22 indicate very good agreement of the simulation results with the existing DSM calculations of AISI (2016) for local buckling resistance based on using the cross-section plastic moment capacity, with average DSM/simulation results of 0.985 and a standard deviation of 0.042. This further confirms validity of the numerical simulation model as well as assumptions of initial imperfections.

4.2.2 Assessment of DSM for local buckling of beams at non-uniform elevated temperatures

Figure 23 compares failure modes of a typical beam with various non-uniform elevated temperature distributions. Due to temperature gradient, the neutral axis under bending moves towards the cooler flange. Since only the depth of the section above the neutral axis is in compression thus liable to local buckling, increasing temperature gradient of the section reduces the depth of local buckling, as shown in Figure 23.

In addition to the local buckling failure mode shown in the above figures, Figure 24 shows an additional failure mode characterised by web local buckling and flange distortion. This failure mode is associated with large temperature gradients (half web depth and full web depth interior insulation) at a low load ratio (0.3). This occurs when the lower flange temperature is very high and thus retaining very little resistance, forcing the neutral axis to move further towards the cooler flange.

When calculating elastic critical buckling loads at elevated temperatures, it was found that the critical buckling loads of beams with high thermal gradients (half insulation and full insulation) fluctuated between different load ratios with some values even higher than the critical buckling load at ambient temperature. This is attributed to the shift of centroid of modulus of elasticity of the cross-section under significant thermal gradient.

Figure 25 plots beam local buckling resistance – slenderness relationship for all the non-uniform temperature cases, and compares the simulation results with the original AISI (2016) DSM equations using plastic moment capacity.

The results in Figure 25 indicate that it is still suitable to use DSM to calculate beam local buckling resistance with non-uniform temperature distribution in the cross-section because all the simulation results are within a reasonably narrow band.

Figure 25 suggests that the original AISI (2016) equations based on plastic moment capacity are suitable, giving DSM/simulation ratio result with average of 0.986 standard deviation of 0.106. Furthermore, results in Figure 25 show that some resistances are greater than the plastic moment capacity of the beams. This is attributed to some strain hardening effect beyond yield. Hence, for design purpose, the plastic moment capacity of the section should be considered as the ultimate capacity.

4.2.3 Assessment of DSM for distortional buckling of beams at ambient temperature

Figure 26 shows typical ultimate failure mode at ambient temperature. The figure shows that the numerical simulation models failed in distortional buckling mode as intended.

Table 4 lists all the simulation results and Figure 27 presents all the simulation results for ambient temperature, plotted to show beam bending resistance/yield moment ratio – beam slenderness relationship. To calculate the beam slenderness, the beam critical elastic buckling load is determined using ABAQUS.

The results in Figure 27 indicate very good agreement between the simulation results with the existing AISI (2016) DSM calculations for distortional buckling resistance based on using the elastic section capacity, with an average of DSM/simulation results of 0.990 and a standard deviation of 0.034. This further confirms validity of the numerical simulation model as well as assumptions of initial imperfection. The simulation results of bending resistance at low slenderness are slightly higher than 1 due to using the first yield bending moment capacity (M_y) of the cross-section.

At non-uniform elevated temperatures, it is more convenient to use the plastic moment capacity of the cross-section. Hence the results in Figure 27 are replotted in Figure 28 based on using the plastic moment capacities of the beam cross-sections.

Obviously, the existing AISI (2016) DSM equations based on the plastic moment capacity of cross-section overestimate the member resistance. To improve accuracy, it is necessary to derive a set of modified equations for use with plastic moment capacity of the cross-section. They are given in equations 1 and 2. Using the modified equations, the average of DSM result/simulation result ratios is 1.005 and the standard deviation is 0.049.

$$M_{nd} = M_p \qquad \text{for } \lambda_d \leq 0.4 \qquad \text{Equation 1}$$

$$M_{nd} = \left[0.85 - 0.18\left(\frac{M_{crd}}{M_p}\right)^{0.45}\right]\left(\frac{M_{crd}}{M_p}\right)^{0.45} M_p \qquad \text{for } \lambda_d > 0.4 \qquad \text{Equation 2}$$

4.2.4 Assessment of DSM for distortional buckling of beams at non-uniform elevated temperatures

Figure 29 compares failure modes of a typical beam at various non-uniform elevated temperatures. It could be seen that all the beams failed in distortional buckling mode.

The results in Figure 30 indicate that it is suitable to use DSM to calculate beam distortional buckling resistance with non-uniform temperature distribution in the cross-section because all the simulation results are within a reasonably narrow band. The results also suggest that the modified DSM equations (equations 1 and 2), are applicable giving mostly safe results. The average of modified DSM/simulation result ratios is 0.923 and the standard deviation is 0.081. As previously explained under local buckling, the results for FEM greater than M_p are a result of strain hardening.

5 Partial safety factors for resistance

According to Zhang *et al*. (2014), the probability of failure (P(f)) of a structure may be expressed as follows:

P(f)=P(f/flashover)P(flashover/fire occurring)P(fire occurring) Equation 3

Fire resistant design of structures is usually based on the assumption of flashover fire. The probability of structural failure under flashover fire is expressed as follows:

P(f/flashover) =P(f)/{P(flashover/fire occurring)P(fire occurring)} Equation 4

If reliability of the structure under fire attack is the same as under ultimate limit state design of the structure at ambient temperature, then the same reliability index of 3.8 should be achieved, corresponding to an overall probability of structural failure of 7.23×10^{-5}. The values are based on recommendations of CEN (2002) for general building design.

The probabilities P(flashover/fire occurring) and P(fire occurring) vary depending on many factors. Accordingly, the acceptable level of P(f/flashover) changes for the same overall probability of structural failure of 7.23×10^{-5}. This implies that there is no single acceptable value of the conditional probability P(f/flashover).

Following Maraveas *et al*. (2017), partial safety factors for structural resistance are calculated for the conditional probability P(f/flashover) of 0.1, 0.01 and 0.001 respectively.

Figure 31(a) shows percentage distribution as a function of DSM calculation/simulation result ratios for all members under local buckling with non-uniform heating, indicating a normal distribution while Figure 31(b) shows that for distortional buckling mode. The overall average of the ratios is 0.986 and the standard deviation is 0.106. Using these values, the partial safety factors for conditional probabilities P(f/flashover) of 0.1, 0.01 and 0.001 can be calculated to be 1.160, 1.333 and 1.498 respectively.

A similar exercise was carried out for distortional buckling with non-uniform temperature distribution in the cross-section. The partial safety factors for conditional probabilities P(f/flashover) of 0.1, 0.01 and 0.001 can be calculated as 1.127, 1.256 and 1.372 respectively

6 Conclusions

This paper has presented the results of an extensive set of numerical parametric study to investigate the behaviour and resistance of transversely loaded cold-formed thin-walled steel beams at ambient and elevated temperatures with non-uniform temperature distributions. The parameters covered different section sizes, slenderness, steel grades and load ratios. All the steel members failed under local and distortional buckling modes as intended. The assessment is based on simulation results generated using an ABAQUS model that was validated against experimental test results for both heat transfer and structural analysis at elevated temperatures. It has been found that a simplified temperature distribution can be used to represent the much more complex non-uniform temperature distribution in thin-walled sections exposed to fire from one side. In the simplification for sections with interior insulation, it is assumed that on either the hot or cold side, the flange/lip temperatures are the same, while temperature distribution in the web is bi-linear with the end temperatures equalling to the hot flange/lip and cold flange/lip temperatures, intersected at the position of interior insulation. For sections without insulation, a linear assumption is used in the web.

The numerical simulation results are used to assess applicability of the direct strength method (DSM). The main findings of this study are:

Direct Strength Method for local buckling:

1. For steel beams at ambient temperature, the numerical load carrying capacities of the beams agree well with calculation results using the existing AISI (2016) DSM equations, based on plastic moment capacity.

2. For steel beams with non-uniformly distributed elevated temperature in the cross-section, the AISI (2016) equations based on plastic moment capacity can be used.
3. To achieve conditional probabilities of P(f/flashover) of 0.1, 0.01 and 0.001, the partial safety factors for structural resistance for non-uniformly heated members are 1.160, 1.333 and 1.498 respectively.

Direct Strength Method for distortional buckling:

4. For steel beams at ambient temperature, the numerical load carrying capacities of the beams agree well with calculation results using the existing AISI (2016) DSM equations. However, if the AISI (2016) DSM equations are applied using plastic moment capacity of the cross-section, the AISI (2016) DSM calculation results overestimate the simulation results. Since it is preferable to use plastic moment capacity, a set of modified DSM equations have been proposed.
5. For steel beams with non-uniformly distributed elevated temperature in the cross-section, the modified DSM equations in (4) are sufficiently accurate.
6. To achieve conditional probabilities of P(f/flashover) of 0.1, 0.01 and 0.001, the partial safety factors for structural resistance for non-uniformly heated members are 1.127, 1.256 and 1.372 respectively.

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Figure Captions

Figure 1. Floor joist arrangements with fire protection on both sides, (a) floor joist arrangement using lipped channel sections, (b) floor joist arrangement using hollow flange sections

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Table Captions

Table 1. Mechanical properties and coefficient of thermal expansion at elevated temperatures

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Table 3. Local buckling of transversely loaded cold-formed steel beams at ambient temperature

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