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## Fundamental Behaviour of Cold-formed Thin-walled Steel Structures at Elevated Temperatures

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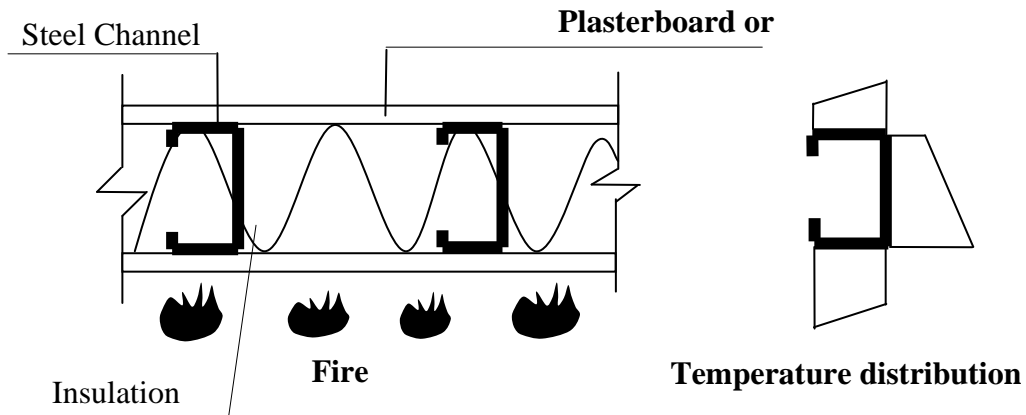
**Abstract.** Under fire conditions, cold-formed steel loses its strength faster than hot-rolled steel. This is due to enhancement in the strength of cold-formed steel at ambient temperature, but this enhancement diminishing at high temperatures. The loss of strength in cold-formed steel may be 20% higher by comparison with hot-rolled steel. Hot-rolled steel members are generally thick and fail either by local or global buckling (e.g. flexural buckling in column and lateral torsional buckling in beams). In comparison, cold-formed thin-walled members are more prone to local buckling and distortional buckling. With regard to global buckling, thin-walled columns can fail in combined local buckling, distortional buckling with torsional or torsional-flexural buckling, in addition to flexural buckling

**Keywords:** steel; cold-formed; thin-walled; high temperature; fire; buckling.

### 1. Introduction

Increases in temperature of a steel section depend on its section factor, which is a measure of the fire exposure area to the volume being heated. Due to small thickness, the section factor of a thin-walled steel member is high and its temperature increase is much more rapid than in a thicker hot-rolled member.

Thin-walled members usually form part of a wall or ceiling and are protected by planar systems (see Figure 1). This type of protection can induce a severe temperature gradient in the depth direction. Furthermore, since thin-walled members lose heat rapidly to the surrounding, a temperature gradient may also be present in the width direction.



**Figure 1.** Temperature distributions in a wall panel exposed to fire attack on one side

**2. Existing investigation of thin-walled steel structures in fire**

Although K.H. Klippstein [1] conducted some research studies on cold-formed steel studs at elevated temperature in the 1970's, more comprehensive works only started after 1990. A number of the studies have concentrated on experiments of light weight steel framed walls [2,3,4] or, tests and numerical studies on cold-formed steel members under uniform high temperatures [5,6].

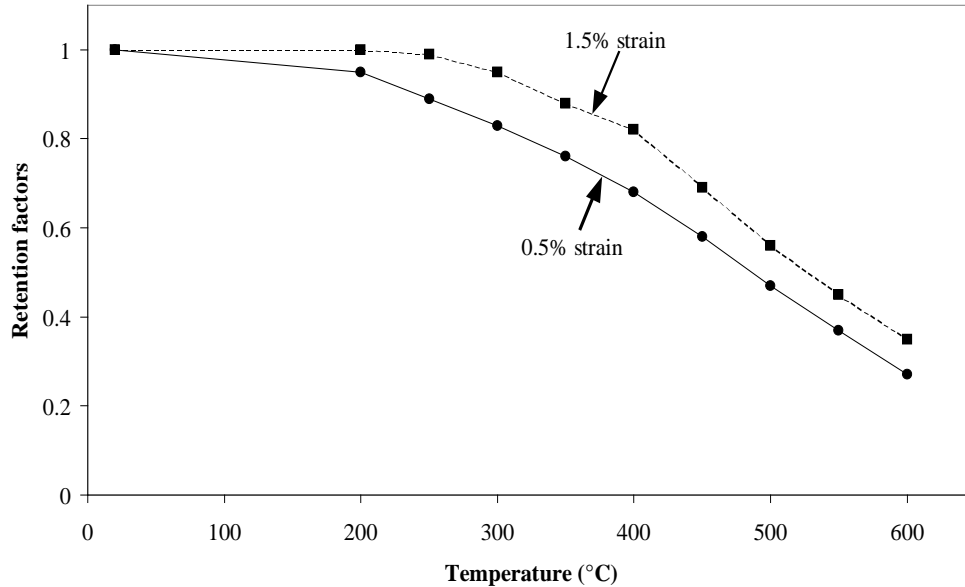
**2.1 Material properties of cold-formed steel at elevated temperatures**

**2.1.1 Stress-strain relationships**

To understand the behaviour of cold-formed thin-walled steel structures in fire, it is necessary to have available information about the mechanical properties of this type steel at elevated temperature.

**Table 1.** Retention factors for stress-strain relationships of cold-formed steel at elevated temperatures

Steel temperature T	Retention factor for elastic modulus	Retention factor for yield strength $k_{p0.2,T} = f_{p0.2,T} / f_{yb}$	Proposed retention factor for yield strength $k_{p2,T} = f_{p2,T} / f_{yb}$
20°C	1	1	1
100°C	1	1	0.97
200°C	0.9	0.89	0.932
300°C	0.8	0.78	0.895
400°C	0.7	0.65	0.857
500°C	0.6	0.53	0.619
600°C	0.31	0.30	0.381
700°C	0.13	0.13	0.143
800°C	0.09	0.07	0.105
900°C	0.0675	0.05	0.067
1000°C	0.0450	0.03	0.029



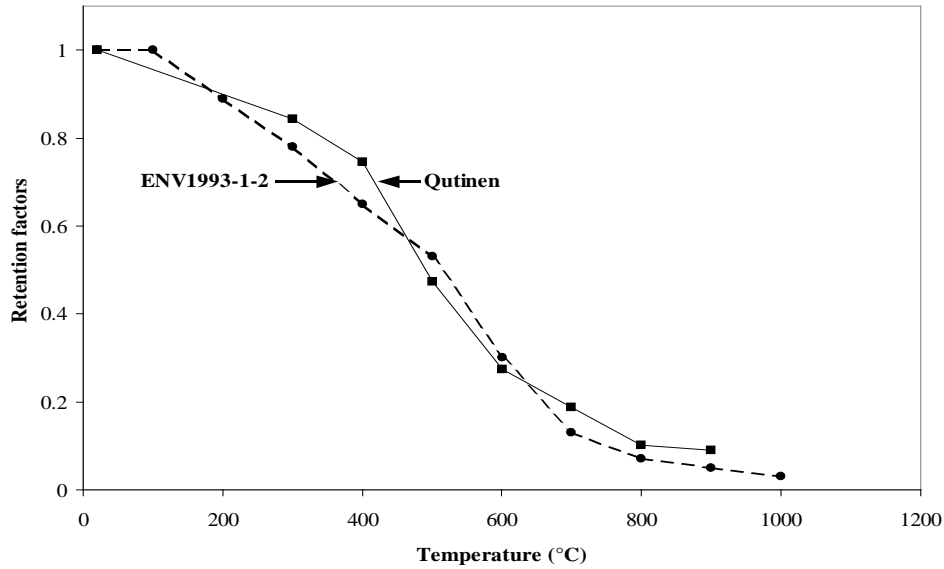
**Figure 2.** Strength retention factors at the 0.5% and 1.5% total strains [5]

ENV1993-1-3 has recently introduced cold-formed, thin-walled steel structures within its scope. This is done by introducing the strength retention factors in Table 1 and Figure 2 into the design equations in ENV1993-1-2 (CEN 2001) for hot-rolled steel structures. The retention factors for material properties of cold-formed steel in ENV1993-1-2, included yield strength and elastic modulus as functions of temperatures, are given in Table 1.

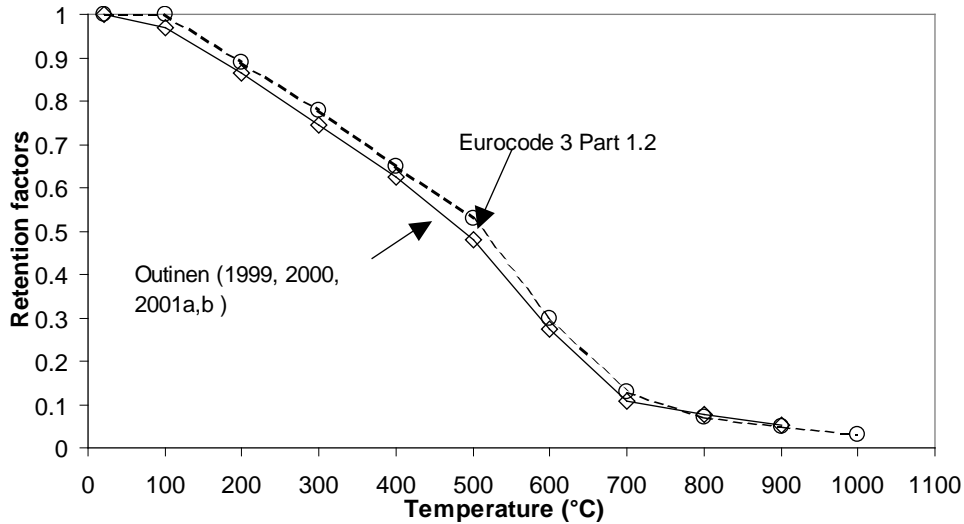
The test results of Sidey and Teague [7] are shown in Figure 2 as ratios of the steel stresses at the 0.5% and 1.5% total strains to the steel yield stress at ambient temperature. These results were used by Lawson [5] to derive limiting temperatures for cold-formed thin-walled steel members.

Outinen [8,9,10] carried out a large number of transient and steady state tensile coupon tests of cold-formed steel at elevated temperatures and reported stress-strain relationships for S355, S420M and S350GD+Z steel in the temperature range 300°C - 600°C and 700°C-950°C. He observed that there were large differences in the results between two different types of tests. This is in agreement with the test results of Kirby and Preston [11] on hot-rolled steel. He also found that the behaviour of the mechanical properties of cold-formed steel is different for different grades of cold-formed steel and the increase in strength due to cold-forming appears to remain at elevated temperature. The  $\sigma$ - $\epsilon$ ,  $f_y$  and  $E$  of grade S355, S420M, S350GD+Z steel in the temperature range 300-950°C have been given. Their measured yield strength for S350GD+Z is quite different from the ENV1993-1-2 recommendation, and they proposed new values of the retention factors, as shown in Table 1, which are based on a total strain of 2%. Figure 3 shows a comparison of the strength retention factors between ENV1993-1-2 and Outinen [8, 9, 10] steady state tests results of S350GD+Z steel based on 0.2% proof stress. It can be seen that the Outinen steady state test results give slightly higher results at temperatures lower than 450°C and slightly lower results at temperatures between 450°C and 650°C. Figure 4 shows a comparison of the strength retention factors between ENV1993-1-2 and Outinen [8, 9, 10] transient state tests results of S350GD+Z steel based on 0.2% proof stress. This is a little different from steady state test results. Outinens transient state test results give slightly lower results at all high temperatures. Outinen gave the same results as in ENV1993-1-2 for the stiffness retention factor and these are the same as for hot-rolled steel.

The cold-forming process used in the fabrication of lightweight steel members leads generally to an increase of the effective yield strength. In open cross-sections, such as C-sections, cold-forming is concentrated in the corners of the cross-section and the yield strength of the plate parts is not greatly influenced by the forming process [12, 13, 14, 15, 16]. Moreover, the influence of residual stresses is smaller at high temperatures.



**Figure 3.** Comparison of strength retention factors for the 0.2% proof stress between ENV1993-1-2 and Outinen steady state test results of S350GD+Z steel



**Figure 4.** Comparison of strength retention factors for the 0.2% proof stress between ENV1993-1-2 and Outinen transient state test results of S350GD+Z steel

### 2.1.2 Thermal expansion

The free thermal expansion of steel is relatively independent of the type of steel [17]. The value can be taken either as:

$$\frac{\Delta l}{l} = 0.4 \times 10^{-8} T^2 + 1.2 \times 10^{-5} T - 3 \times 10^{-4} \quad (1)$$

or, with little loss of accuracy as

$$\frac{\Delta l}{l} = 1.4 \times 10^{-5} (T - 20) \quad (2)$$

Lie [18] published the coefficient of thermal expansion at different temperatures as:

$$\frac{\Delta l}{l} = (0.004T + 12) \times 10^{-6}, \text{ for } T < 1000^\circ\text{C} \quad (3)$$

ENV1993-1-2 gives a slightly different version as follow:

$$\frac{\Delta l}{l} = -2.416 \times 10^{-4} + 1.2 \times 10^{-5} T + 0.4 \times 10^{-8} T^2, \text{ for } 20^\circ\text{C} \leq T < 750^\circ\text{C} \quad (4)$$

$$\frac{\Delta l}{l} = 1.1 \times 10^{-2}, \text{ for } 750^{\circ}\text{C} \leq T \leq 860^{\circ}\text{C} \quad (5)$$

$$\frac{\Delta l}{l} = 2 \times 10^{-5} T - 6.2 \times 10^{-3}, \text{ for } 860^{\circ}\text{C} < T \leq 1200^{\circ}\text{C} \quad (6)$$

A linear thermal expansion coefficient of  $14 \times 10^{-6}$  per  $^{\circ}\text{C}$  above  $100^{\circ}\text{C}$  is adopted in BS5950 Part [19].

### 2.1.3 Thermal properties

The temperature rise of steel members, which is a result of heat flow, is a function of the thermal conductivity and specific heat of steel. The thermal conductivity of steel may be taken as defined in ENV1993-1-2 or BS5950: Part 8 [20]. ENV1993-1-2 gives a linear relationship between the thermal conductivity of steel and temperatures from  $20^{\circ}\text{C}$  to  $800^{\circ}\text{C}$  as:

$$\lambda_a = 54 - 3.33 \times 10^{-2} T \text{ (W/mK)}, \text{ for } T = 20^{\circ}\text{C} - 800^{\circ}\text{C} \quad (7)$$

For temperatures higher than  $800^{\circ}\text{C}$ , a constant value of  $27.3 \text{ W/mK}$  can be used. In simple calculation methods, the thermal conduction of steel may be assumed to be independent of the steel temperature, giving:

$$\lambda_a = 45 \text{ W/mK} \quad (8)$$

BS5950 Part 8 [19], recommends a value of  $37.5 \text{ W/m} \cdot ^{\circ}\text{C}$ .

The specific heat of steel increases gradually with temperature. A constant value of  $600 \text{ J/kg} \cdot ^{\circ}\text{C}$  is suggested for temperatures below  $600^{\circ}\text{C}$  (Lie, 1992 and Anderberg, 1983).

The following values for specific heat of steel are recommended in ENV1993-1-2:

$$C_a = 425 + 7.73 \times 10^{-1} \theta - 1.69 \times 10^{-3} \theta^2 + 2.22 \times 10^{-6} \theta^3; \text{ (} 20^{\circ}\text{C} \leq \theta < 600^{\circ}\text{C} \text{)} \quad (9)$$

$$C_a = 666 + \frac{13002}{738 - \theta}; \text{ (} 600^{\circ}\text{C} \leq \theta \leq 735^{\circ}\text{C} \text{)} \quad (10)$$

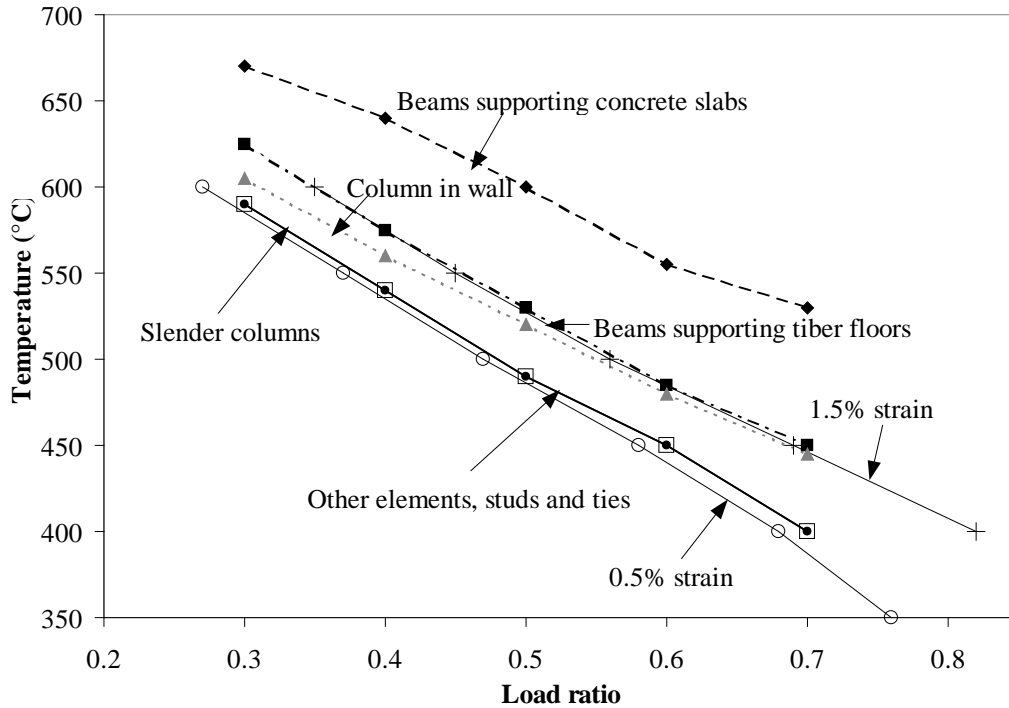
$$C_a = 545 + \frac{17820}{\theta - 731}; \text{ (} 735^{\circ}\text{C} \leq \theta \leq 900^{\circ}\text{C} \text{)} \quad (11)$$

$$C_a = 650; \text{ (} 900^{\circ}\text{C} \leq \theta \leq 1200^{\circ}\text{C} \text{)} \quad (12)$$

BS5950 Part 8 gives a value of  $520 \text{ J/kg} \cdot ^{\circ}\text{C}$ .

### 3. Cold-formed thin-walled steel members at uniform elevated temperature

The SCI [5] have published a design method for cold-formed thin-walled structural members by adopting the limiting temperature method in BS5950 Part 8 [19] for hot-rolled steel members. In general, the limiting temperatures of cold-formed member are lower. For beams, the limiting temperatures are based on the steel strength at 1.5% strain, as opposed to the 2% strength for hot-rolled steel. The column limiting temperatures are based on the steel strength at 0.5% strain, but the cold-formed steel strength at 0.5% strain is lower than that of hot-rolled steel. The strength reduction factors at different strain levels for cold-formed steel at elevated temperatures are given in Table 2. Figure 5 compares the limiting temperatures of different types of cold-formed thin-walled steel structures and the strength retention factors of cold-formed steel in Figure 5 at different temperatures.



**Figure 5.** Limiting temperatures (°C) of different types of cold-formed thin-walled steel structures (Lawson 1993)

**Table 2:** Strength reduction factors for cold-formed steel at elevated temperatures (°C) (Lawson 1993)

Temperature (°C)	200	250	300	350	400	450	500	550	600
0.5% strain	0.95	0.89	0.83	0.76	0.68	0.58	0.47	0.37	0.27
1.5% strain	1.00	0.99	0.95	0.88	0.82	0.69	0.56	0.45	0.35

Ala-Outinen and Myllymaki [21] reported a series of transient-state fire tests on 900mm long RHS 200x200x5 and RHS 150x100x3 rectangular hollow sections under concentric and eccentric compression loading. During all of the tests, the axial load was kept constant and the furnace temperature was controlled to rise from 20°C to 300°C in 3 minutes and subsequently by 10°C/min. They found that the concentrically loaded columns failed in local buckling at the middle of the column, and the eccentrically loaded columns ( $e = 28$  mm) failed in local buckling near the top of the column. They also proposed a design method, which is based on ENV1993-1-3. In their method, the reduced yield strength based on the 0.2% proof strain at ambient temperature and the reduced elasticity according to ENV1993-1-2 are used.

Randy [22] carried out a theoretical study and pointed out that the influence of elevated temperatures on local buckling should be taken into account and the calculation method for effective width according to ENV1993-1-3 (CEN 2001) was accurate enough for elevated temperatures, provided that the yield strength was taken as 0.2% proof stress at elevated temperatures.

O. Kaitila [23,24,25] carried out an imperfection sensitivity studies in lipped channel columns 100x40x15x1 with length  $L=2500$ mm at high uniform temperatures. The columns are assumed to be attached to plasterboards and are designed to be integrated into a wall structure.

Therefore, the columns are assumed to have sufficient restraint against torsional and torsional-flexural buckling. In his modelling, the influence of thermal expansion has not been considered. He found that the magnitude of initial local imperfections has an effect on the compression stiffness of the structure, whereas the magnitude of global flexural imperfection has more influence on the ultimate strength of the structure. A combination of local and global imperfections should be used in ultimate strength simulations of columns under uniform high temperatures. The suitable value for global imperfection is about column length over 500 and for local imperfection is about the width of web over 200.

Lee, Mahendran and M<sup>TM</sup>kel<sup>TM</sup>inen [26] presented the results of 36 steady state fire tests and finite element modelling of the buckling behaviour of thin-walled compression members at elevated temperatures. Four types of lipped channels with 400mm length and pin end supports were studied. The results of the fire tests and finite element analyses were used to determine the plate buckling coefficient  $k_T$  at elevated temperatures. They found that finite element analysis could be successfully used to model the behaviour of thin-walled compression members at uniform high temperatures and the current design rules can be modified to take into account the local buckling effects of thin-walled compression members at elevated temperatures if the reduced yield strength and the reduced elastic modulus were used.

#### **4. Cold-formed thin-walled steel in fire**

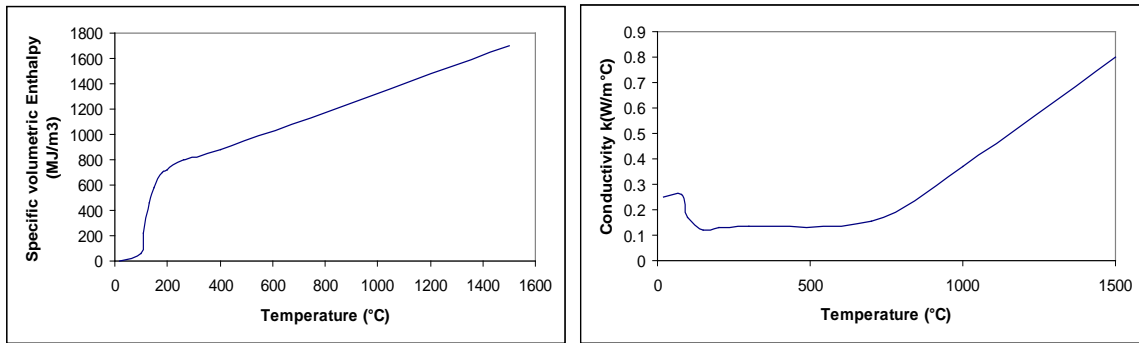
##### **4.1 Temperature distributions of thin-walled panel systems in fire**

If Cold-formed thin-walled steel sections are used as part of a fire resistant construction, they should satisfy the three fire resistant requirements, namely integrity, insulation and stability. Fire resistant barriers play an important role in maintaining building integrity and reducing the spread of fire. The insulation condition requires that the unexposed surface of a construction element does not get ignited due to excessive temperature rise. Whether further ignition on the unexposed surface will occur or not will depend on the materials on the unexposed side and their configuration relative to the construction element. However, current fire resistant regulations specify that the average temperature rise on the unexposed surface should not exceed 140°C [7,11]. The load bearing condition requires the load bearing members to remain stable during the entire fire exposure to satisfy the stability requirement.

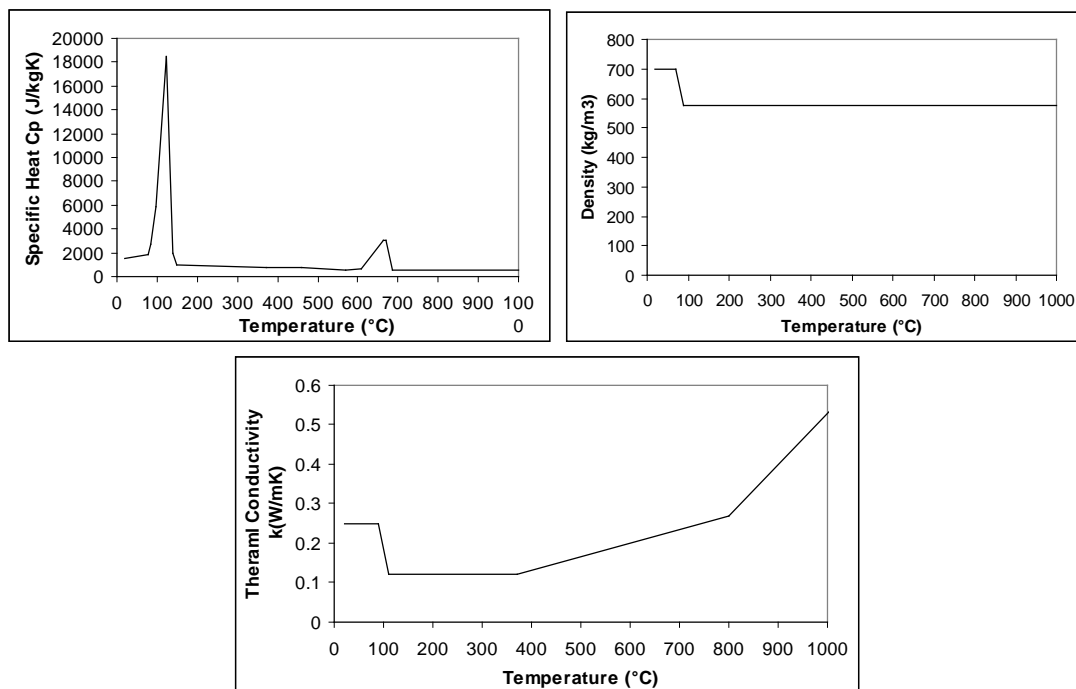
Gerlich [4] reported the results of 3 fire tests on loaded light steel frame walls and carried out a thermal analysis of these three walls using TASEF. He pointed out that the failure of frame walls was determined by the performance of gypsum plasterboard lining, the cold-formed steel properties at elevated temperature and thermal bowing induced deflection. Because the magnitude of thermal bowing will be determined by thermal properties of gypsum plasterboard lining, the thermal properties of gypsum board will affect the result of tests and numerical analysis on temperature distributions in frame walls. The values of specific heat and conductivity in Figure 6 have been used in his thermal analysis. Because TASEF (Sterner and Wickstorm 1990) does not model mass transfer (moisture movement), the predicted temperature results on the unexposed side were lower than the test results.

Cooper [27] developed GYPST, a Fortran Subroutine to simulate the thermal response of steel stud gypsum board assemblies exposed to fire. Two full-scale standard furnace tests were performed to verify predictions from the model. The thermal properties of gypsum board shown in

Figure 7 has been used in his modelling. Good comparisons were achieved between predicted and experimental results.



**Figure 6.** Thermal properties of gypsum board in Gerlich [4]



**Figure 7.** Thermal properties of gypsum board used by Cooper [27]

Sultan, Alfawakhiri and Bnichou [28] presented a one-dimensional heat transfer model for predicting the temperature distribution across loaded and unloaded steel stud wall assemblies with either glass or rock fibre insulation in the wall cavity. Assemblies considered in their study include one unloaded assembly with one layer of gypsum board on one side and two layers on the other side, and three loaded wall assemblies with two layers of gypsum board on each side. In their modelling, moisture migration through the gypsum board was not considered, however, the heat absorbed in the dehydration of the gypsum board was included in the model. Shrinkage of either insulation or the gypsum board due to fire exposure was not considered in the model. After comparing the standard fire tests and numerical predictions, they found that the model provided a reasonable temperature history across the assemblies but some important factors have not been considered in the model, such as forced heat convection, which plays a significant role in the initial 10min to 15 min of the fire test, insulation shrinkage, moisture of gypsum board, all leading to underestimation of predicted results.



Jones [29] reported a comprehensive study of the performance of gypsum plasterboard assemblies, including steel stud walls and timber stud walls. The emphasis of this study was that of the gypsum boards. He pointed out that ablation, cracking and shrinkage of gypsum boards are mechanical characteristics of gypsum plasterboard, which can have significant influences on heat transfer through walls lined with gypsum plasterboard. Unfortunately, numerically simulating ablation, cracking and shrinkage of gypsum boards is too difficult and until now there is no satisfactory resolution to these problems. He suggested that if heat transfer analysis only were used to analysis the performance of gypsum board, different thermal properties of the same gypsum board would be required to achieve the same accuracy between numerical analysis results and test results under the standard fire and natural fire conditions. This is because the different fire expose conditions will generate different temperature gradient and cause different moisture movements, which will seriously affect temperature distributions. Ideally, mass transfer should be considered in heat transfer analysis on steel stud walls and timber stud walls.

#### 4.2 Structural behaviour of cold-formed thin-walled steel in fire

Gerlich [4] reported three full-scale light steel framed wall fire tests in which two 3600mm high wall panels with lipped channel section of 102x51x1.0mm and one 2850mm with unlipped channel 76x32x1.15mm were loaded and then exposed on one side to the standard fire exposure. Substantial temperature gradients in the steel studs were recorded. Local buckling of the steel studs and lateral deflection induced by thermal bowing were observed. In two tests, failure was flexural buckling about the major axis initiated by local buckling of the compression flange between fasteners adjacent to the unexposed lining. The other test's failure mode was by torsional flexural buckling after the unexposed lining failed to provide lateral restraint to the compression flange. This suggests that the non load-bearing gypsum board provided important restraints to the steel studs. He also presented a design model to determine the critical failure temperature of cold-formed thin-walled steel stud. The model is based on the AISI design manual and adopts the reductions in the yield strength and modulus of elasticity of steel given by Klippstein [1]. The proposed design checks are:

$$\sigma = \frac{N_c}{A} - \frac{N_c[e(\Delta T) + e(M)]}{W} \leq f_{y,T} \quad \text{for the hot flange} \quad (13)$$

$$\sigma = \frac{N_c}{A} + \frac{N_c[e(\Delta T) + e(M)]}{W} \leq f_{y,T} \quad \text{for the cool flange} \quad (14)$$

where  $N_c$  is the applied axial compression force,  $A$  is the gross cross-sectional area of the steel stud,  $e(\Delta T)$  is the mid-length deflection due to thermal effects,  $e(M)$  is the mid-length deflection due to bending moment and  $W$  is the elastic section modulus about the stronger axis of the cross-section; and  $f_{y,T}$  is the yield stress on steel at the cold side temperature. This approach does not consider local buckling effects on CF-TW steel studs.

Alfawakhiri and Sultan [30,31] reported the results of six standard fire resistance tests on axially loaded lightweight steel framed (LSF) walls exposed to fire on one side. Each assembly consisted of a single row of galvanized cold-formed steel studs and protected with two layers of fire-resistant gypsum board on each side. Four of these specimens incorporated three types of insulation (glass fibre batts, rock fibre batts and dry blown cellulose) and the other two without any interior insulation. During the early stage of the fire tests, the LSF specimens bowed toward the fire test furnace due to thermal bowing. At the later stage, the LSF specimens without interior insulation continued to bow towards the furnace and failed by compressive crushing on the cold face in the middle of the test specimens, while those with interior insulation reversed direction to bow away from the fire test furnace and failed by compression crushing on the hot face at the locations of the service hole near one end of the assemblies. Because of high temperature gradients in interiorly insulated panels, the results of these fire tests suggest that interior insulation can cause a reduction in the fire resistance of load-bearing LSF walls. Alfawakhiri and Sultan [30,31] have also presented a comprehensive structural model for thin-walled steel wall studs subject to severe heating. In that model, they assumed that flexural-torsional and weak axis buckling failure were prevented by adequate lateral restraints and there was no temperature variation in the vertical direction along the stud but there were temperature gradients across the stud section from

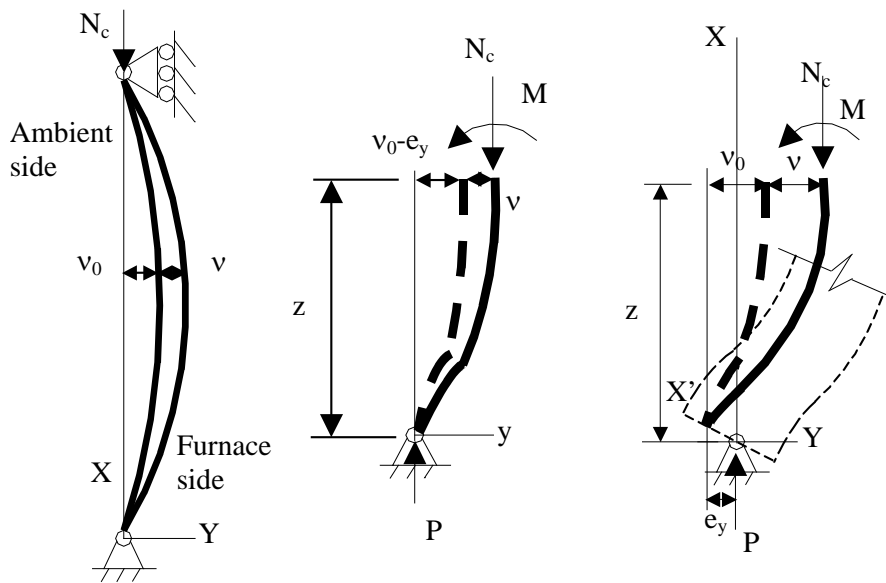
one side to the other. Therefore, under vertical load and non-uniform temperatures, thermal bowing and deflection will be induced, as shown in Figure 8. The total lateral deflection can be calculated as:

$$y(z) = v + v_0 = (\varphi\beta^{-2} - e_y)[\tan(0.5\beta H)\sin(\beta z) + \cos(\beta z) - 1] \tag{15}$$

where,  $\beta = \sqrt{N_c / (EI^*)}$ ;  $\varphi = \alpha\Delta T / b_w$  is the thermal bowing curvature;  $\Delta T$  is temperature difference across the stud section;  $H$  is the stud height;  $I^*$  is elasticity-modulus-weighted moment of inertia of the unreduced stud section about the neutral axis parallel to flanges;  $E$  is the modulus of elasticity of steel at room temperature;  $e_y$  is an eccentricity that is dependent on the non-uniform stiffness distribution at ends, loading condition and bounding condition of the stud and is given as:

$$e_y = (1 - K_R)\varphi\beta^{-2} \tag{16}$$

where  $K_R$  is a reduction coefficient which has a value of 0.6.



**Figure 8.** Buckling of a column due to non-uniform temperature effect [30]

After the lateral deflections have been determined, the following two equations can be used to predict the column ultimate failure load.

$$\frac{E_T}{E} \left( \frac{N_c}{A_e^*} + \frac{N_c[e - y(z)]}{S_{eH}^*} \right) \leq f_{yH} \tag{17}$$

, at supports

$$\frac{E_T}{E} \left( \frac{N_c}{A_e^*} + \frac{N_c y(z)}{S_{eC}^*} \right) \leq f_{yC} \tag{18}$$

, at mid height

where,  $E_T$  is the elastic modulus of steel at temperature  $T_H$ ,  $f_{yH}$  is the yield stress of steel at temperature  $T_H$ ;  $A_e^*$  is the temperature dependent elasticity-modulus-weighted effective stud section area in compression; and  $S_{eH}^*$  and  $S_{eC}^*$  are the temperature dependent elasticity-modulus-weighted effective stud section elastic modulus in bending for compression in the hot flange at supports and cold flange at mid-height, respectively. This approach does not consider column global buckling.

### 5. Conclusions

This chapter has briefly summarized the existing relevant studies in cold-formed thin-walled steel structure areas, including the studies of the behaviour of cold-formed thin-walled steel structures at ambient and high temperatures. It can be found that although the behaviour of cold-formed thin-walled steel structures at ambient temperature, including local buckling, distortional buckling, global buckling and shear buckling have been well understood and suitable design methods existed, there are only sporadic research studies of cold-formed thin-walled steel

structures at high temperatures. Because of the lack of a systematic research study on cold-formed thin-walled steel structures in fire a suitable design procedure can not be drawn, which impedes the adoption of cold-formed thin-walled steel structures in the construction market.

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