STRUCTURAL BEHAVIOUR OF UNRESTRAINED COMPOSITE TRUSS SYSTEMS IN FIRE

SK Choi[1], IW Burgess[2] and RJ Plank[3]

ABSTRACT

This project is aimed at investigating the fire performance of long-span composite trusses, widely applied in long-span multi-storey floor construction, using the Sheffield FE package Vulcan. The basic layout of the composite trusses used throughout this project is composed of a series of individual elements: top & bottom chord members, diagonal bracing and a profiled steel deck with a concrete topping. The numerical analyses of the unprotected composite truss model in a standard ISO834 fire are conducted on the basis of the time-temperature relationship of each component, obtained using the Eurocode 4 formula and experiments. Using the results from the numerical work, critical members are modified and protected by a thin-film intumescent coating to improve the performance of the models for a 60-minute fire resistance rating in the standard ISO834 fire.

Key Words: *Composite truss, Fire resistance, Fire protection*

1 MODELLING

A standard model of a Warren truss, 15.0m span and 3.0m spacing, composite with a concrete slab and profiled steel decking has been used for this study (Figs. 1a and 1b). The model has been designed for ultimate strength and deflection criteria at ambient temperature. The depth of the truss, 750mm, is based on a typical span/depth ratio for a simply supported structure (span/20). The chord members (Figs. 1d and 1f) were selected from the S350 cold-formed sections provided by Metsec Ltd. The steel decking (Fig. 1e) acts as permanent formwork to the underside of the lightweight C30 concrete slab. A142 anti-crack steel mesh reinforcement is placed in the concrete slab. Loading at the fire limit state for the proposed composite truss, which is a typical office floor loading factored according to BS5950:Pt8[1], is 6.73kN/m^2.

The geometry of the truss is assumed to be such that there is no eccentricity at the connections (represented as centroidal lines in Fig. 1c). All transverse loading is applied at the connections of the top chord and bracing members. The model is assumed to be simply supported at the top chord and perfectly straight.

[1] Postgraduate student, Dept. of Civil and Structural Eng., Univ. of Sheffield, Sheffield, UK
[2] Professor, Dept. of Civil and Structural Eng., Univ. of Sheffield, Sheffield, UK
[3] Professor, School of Architectural Studies, Univ. of Sheffield, Sheffield, UK

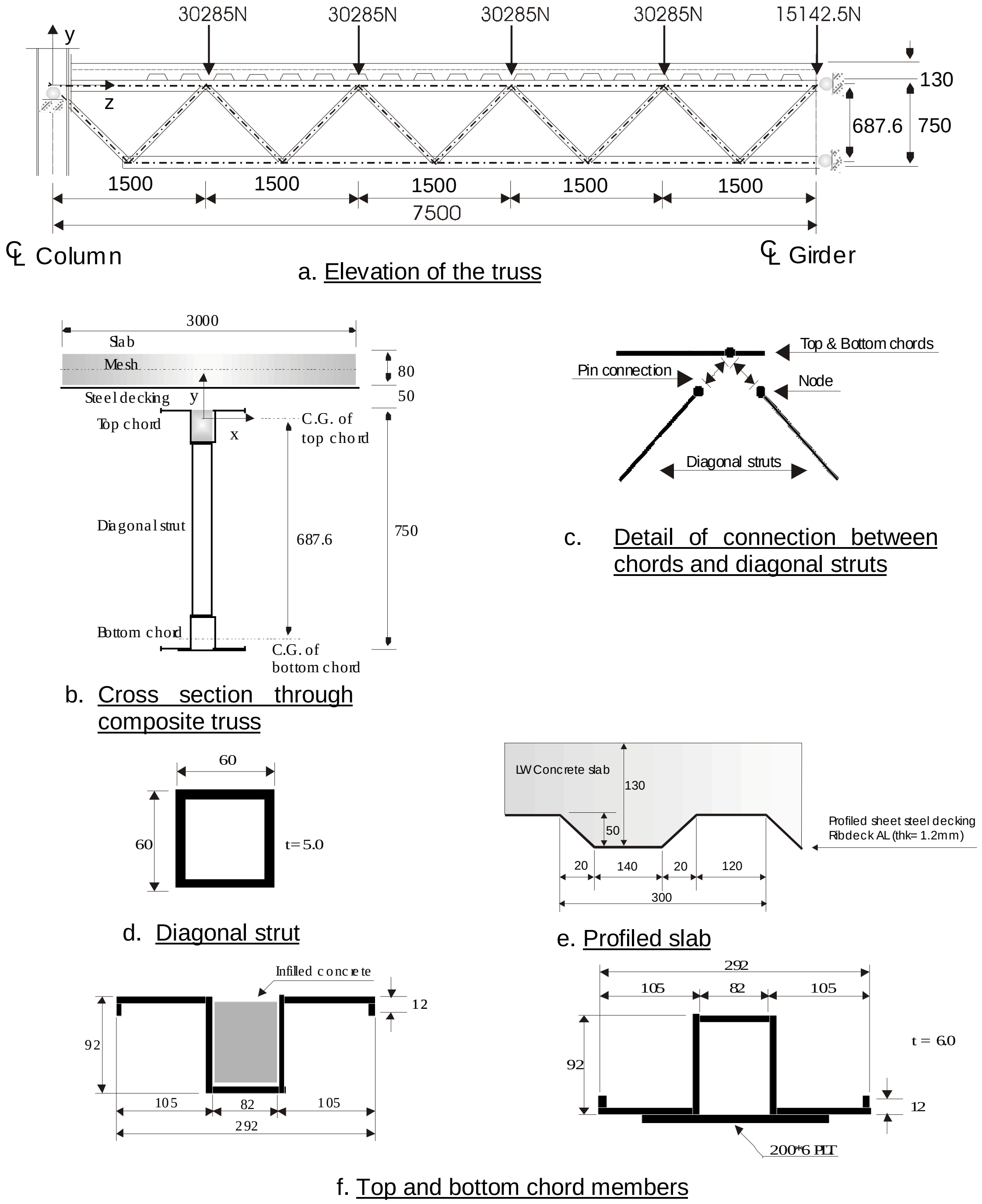


Fig. 1 General layout of the composite truss model (dimensions in mm)

In such a composite truss at ambient temperature, member behaviour is dominated by axial forces, which can be determined by a standard pin-jointed truss analysis. The proposed model is more realistically modelled by choosing continuous top & bottom chords and pin-ended diagonal struts (Fig. 1c). A rigid connection is not recommended for most directly-welded trusses as it exaggerates bracing member end-moments.

Throughout this work the material properties of steel and concrete at elevated temperatures are calculated by using formulae given in EC3:Pt1.2[2] and EC4:Pt1.2[3]. No specific stress-strain relationship for lightweight concrete is given in this code. Hence, the stress-strain curves for NWC in EC4:Pt1.2 is used.

In order to assess simply the stability condition of the members in the composite truss at elevated temperatures, simple formulae are have been proposed for the load ratio of each member. A pin-ended diagonal strut in compression can be considered as simply supported. Its load ratio R_c may be represented by:

$$R_c = \frac{F_f}{A_g\, p_c} \tag{1}$$

where F_f is the axial load at the fire Limit State

A_g is the gross area

p_c is the compressive strength according to BS5950:Pt1[4]

BS5950:Pt8 recommends a 0.5% strain limit for compressive members on the basis of experimental data[5,6,7]. Using the Strength Reduction Factor corresponding to 0.5 % strain, $SRF_{0.5}$, according to BS5950:Pt8 a modified load ratio for the pin-ended columns, R_{cm}, may be represented by:

$$R_{cm} = \frac{F_f}{A_g\, p_c\, SRF_{0.5}} \tag{2}$$

The bottom chord and diagonal struts in tension can be classified as tension members, as the bending moments in the bottom chord may be ignored at large deflections in fire conditions. Tensile resistance is proportional to the strength reduction of the material. Hence, the load ratio of a can be tension member, R_t, represented as

$$R_t = \frac{F_f}{A_g\, p_y} \tag{3}$$

where p_y is the design strength of steel

BS5950:Pt8 adopts a 1.5% strain limit for non-composite members in bending. This is based on the experiments of Kirby and Preston[8], which show that a local strain of 2-3% can be attained under a range of stress conditions at the centre of the tension flange when the deflection reaches a limiting value of *span/30*. The modified load ratio for tension members, R_{tm}, can be expressed by using the strength reduction factor, $SRF_{1.5}$, corresponding to 1.5% strain as

$$R_{tm} = \frac{F_f}{A_g\, p_y\, SRF_{1.5}} \tag{4}$$

2 THE STANDARD MODEL SUBJECT TO UNIFORM ELEVATED TEMPERATURES

The standard model, subject to a uniform elevated temperature, was numerically analysed using VULCAN. Axial forces for each member at ambient temperature are illustrated in Fig. 2. Double black and white arrows indicate compression and tension respectively. Single black vertical arrows represent resultant shear forces.

The critical elements of the standard model can be identified at ambient temperature by examining the load ratios in the members. The peak value of load ratio under tension occurs in the mid-span member of the bottom chord, 6.75-7.5m from the support. For compression it is the first compressive diagonal strut at 0.75–1.5m from the support. These members may also be the critical elements under elevated temperature conditions.

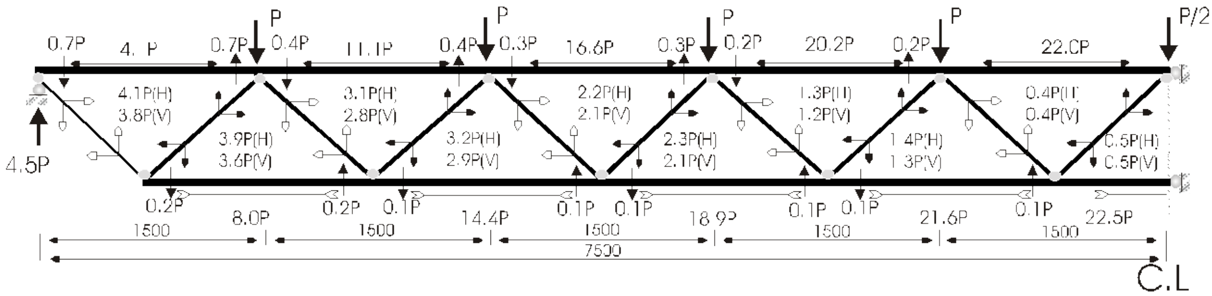
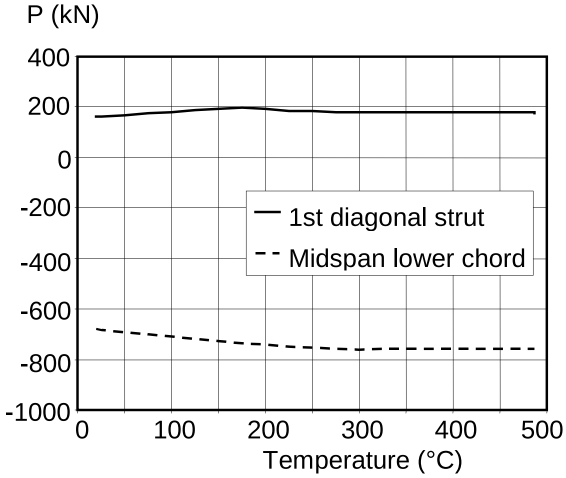


Fig. 2 Internal forces in the standard model at ambient temperature

Under uniform elevated temperatures, the internal forces of the critical members increase by about 10–11% as the temperature increases to 300°C. This is caused by the difference in thermal expansion between the steel chords and the lightweight concrete slab (Fig. 3). Using Equations 1 to 4, the variation of load ratio and modified load ratio of the critical elements can be plotted against temperature in Fig. 4. This does not show clearly which of the two elements initiates a failure at about 487°C.



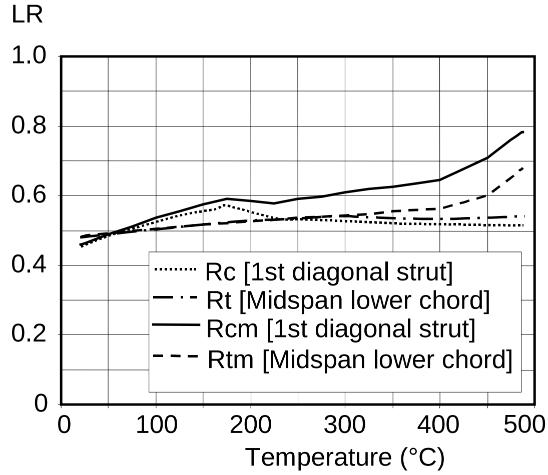


Fig. 3 Variation of axial forces of critical Members at elevated temperature

Fig. 4 Variation of (modified) load ratio of critical members at elevated temperature

The first compressive diagonal strut can be seen to have the highest value of modified load ratio, so its behaviour was further examined. Failure temperatures of pin-ended RHS columns, representing the diagonal strut, for a range of slenderness ratios and load ratios, were determined by VULCAN. Imperfections and design loads were provided by strut curve (a) from Annex C of BS5950:Pt1. Comparing this to the first compressive diagonal strut in the standard model with respect to failure temperatures, slenderness and load ratios, it is verified that this member causes the failure of the standard model (Fig. 5). The first compressive diagonal strut buckles at a slightly higher temperature than the pin-ended column as it is not assumed to be subject to as high an imperfection value as the pin-ended column.

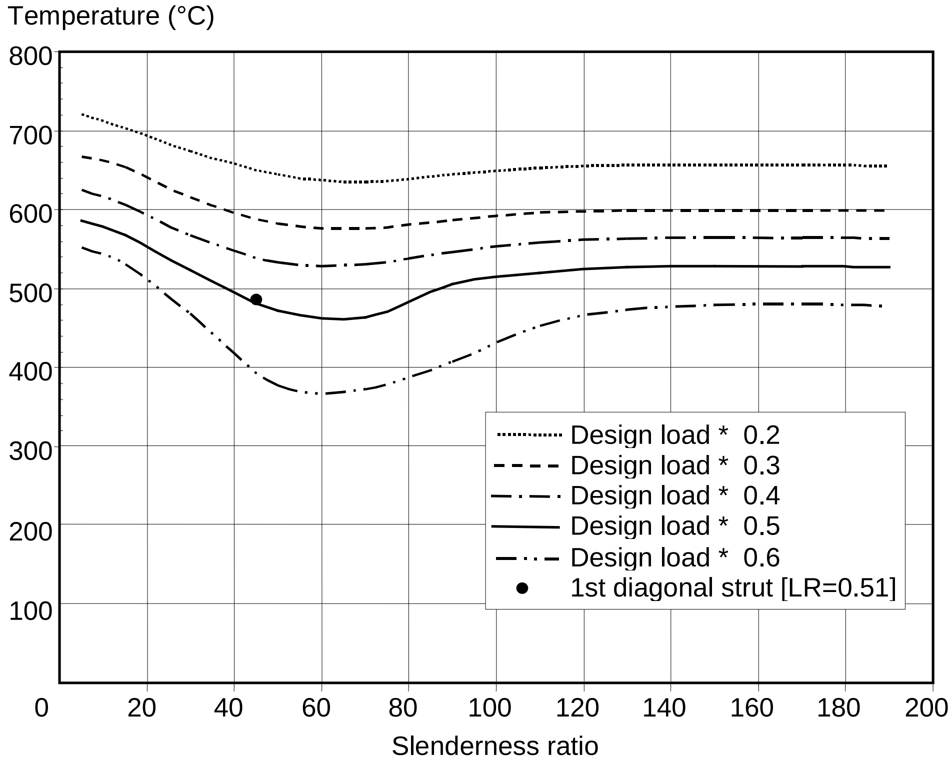


Fig. 5 Comparison between failure temperatures of the first compressive diagonal strut and pin-ended struts; load ratios are based on BS5950:Pt1.

3 UNPROTECTED COMPOSITE TRUSS SYSTEM SUBJECT TO THE STANDARD ISO834 FIRE

The structural behaviour of unprotected composite trusses, subjected to a standard ISO834 fire, with the same set of top and bottom chords as adopted in the standard model and various sizes of diagonal struts was numerically investigated using VULCAN.

3.1 Temperature assessment

The temperature development of each steel member in the unprotected composite truss, subjected to a standard ISO 834 fire, was calculated using a formula provided by EC4: Pt1.2. The temperature increase, $\Delta\theta_a$, of various parts of a bare steel beam during the time interval, Δt, may be determined by following formula [3, 9] :

$$\Delta\theta_a = \frac{\alpha_c + \alpha_r}{C_a \rho_a} \cdot \frac{A}{V} \cdot \left(\theta_t - \theta_a\right) \cdot \Delta t \tag{5}$$

Where α_c is the coefficient of convective heat transfer (25W/m^{2}K for cellulosic fires)

α_R is the coefficient of radiative heat transfer

$$\left[= \Phi \left(\frac{5.67 \times 10^{-8} \varepsilon_{res}}{\theta_t - \theta_a} \right) \left(\left(\theta_t + 273\right)^4 - \left(\theta_a + 273\right)^4 \right) \right]$$

in which Φ is the configuration factor (conservatively taken as 1.0)

ε_{res} is the resultant emissivity of the fire compartment and the surface

C_a is the specific heat of steel (600 J/kgK)
ρ_a is the density of steel (7850kg/m^3)
A is the exposed surface area of the steel cross section per unit length (m^2/m)
V is the volume of the steel cross section per unit length (m^3/m)
θ_t is the average gas temperature during the interval Δt (^{o}C)
θ_a is the steel temperature at the end of the interval Δt (^{o}C)

The section factor *A/V*, may be used to determine the heating rate of each steel member, or even parts of cross-sections. Members with lower section factors are the more massive, having low heated perimeter and high mass, and therefore show the lower rates of temperature rise. For the top chord in the truss the exposed surface area, *A,* is reduced because of the slab and the concrete infill. Values of section factors for the standard top and bottom chords and a range of diagonal struts are tabulated in Table 1.

Table 1 Section factors for each of the members

Section		A [mm^2]	V [mm^3]	A/V [m^{-1}]
Top Chord		500	2822	177.2
Bottom Chord		792	4022	196.9
Diagonal Strut	60*60*5.0	240	1090	220.2
	60*60*6.3	240	1330	180.5
	70*70*6.3	280	1590	176.1
	80*80*6.3	320	1840	173.9
	90*90*6.3	360	2090	172.3
	80*80*8.0	320	2270	141.0
	90*90*8.0	360	2590	139.0
	60*60	240	3600	66.7
	80*80	320	6400	50.0

Using Equation 5 iteratively, the temperature development of the top and bottom chords and diagonal struts (SHS 60*60*5 and solid section 80*80) under standard fire exposure were calculated (Fig. 6).

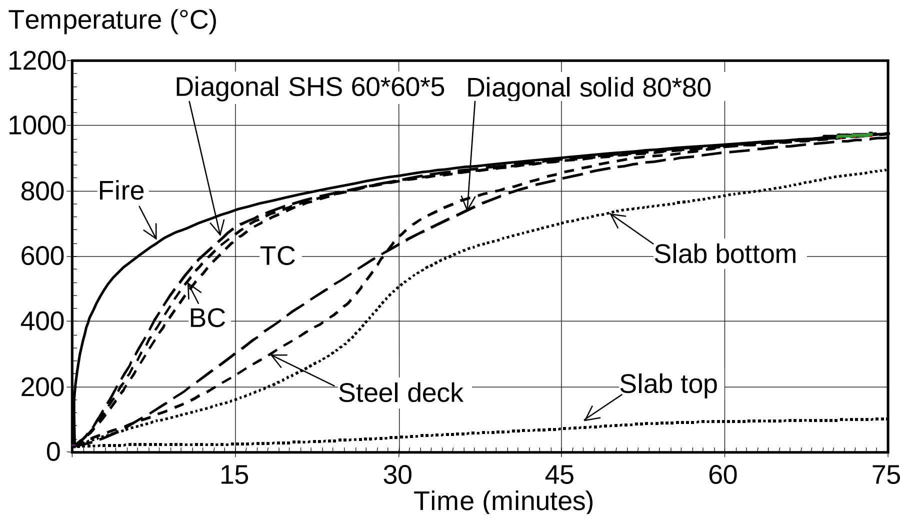


Fig. 6 Temperature development of composite truss in the standard fire.

The section of the concrete slab was numerically modelled by laminating it into 10 layers through its depth. Each layer was further divided into 20 segments. The temperature development of each layer was assessed based on experimental data[10]. The time-temperature curves of top and bottom layers of the slab are included in Fig. 6.

3.2 Parametric study

The structural behaviour of an unprotected composite truss of the type discussed has been investigated with various sizes of diagonal strut, subjected to a standard fire. The results are plotted in Fig. 7 as vertical deflection against the temperature of the bottom chord member for different diagonal strut cross-sections. It can be seen that by modifying the size of the diagonal strut the bottom chord failure temperature of the unprotected model can be improved to 670°C, which is reached after about 15 minutes in the standard fire.

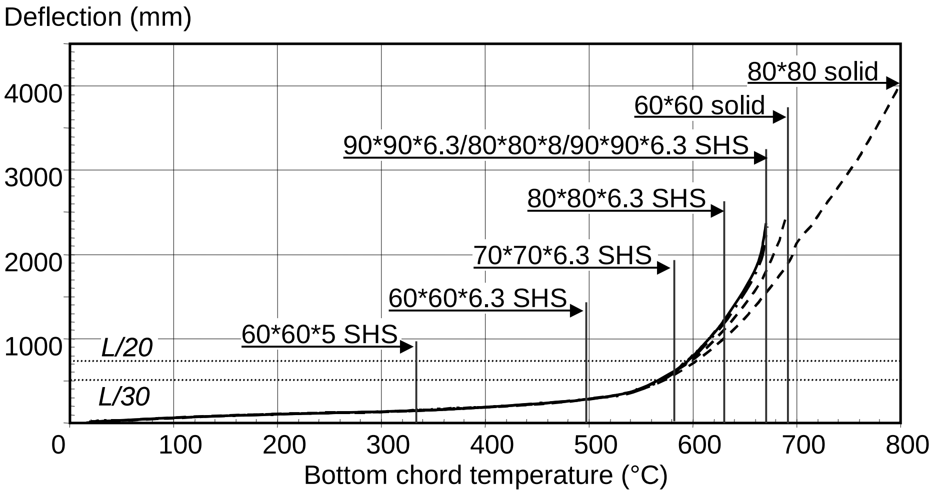


Fig. 7 Vertical deflection at the mid-span of the top chord with various types of diagonal strut

Fig. 8 shows that failure temperatures of the modified trusses increase with the cross-sectional area of the first diagonal strut up to a bottom chord temperature of 670°C. This suggests that the first diagonal strut is the critical element if its section size is below SHS 90*90*6.3 (2090mm^2). For larger struts the mid-span bottom chord member is critical.

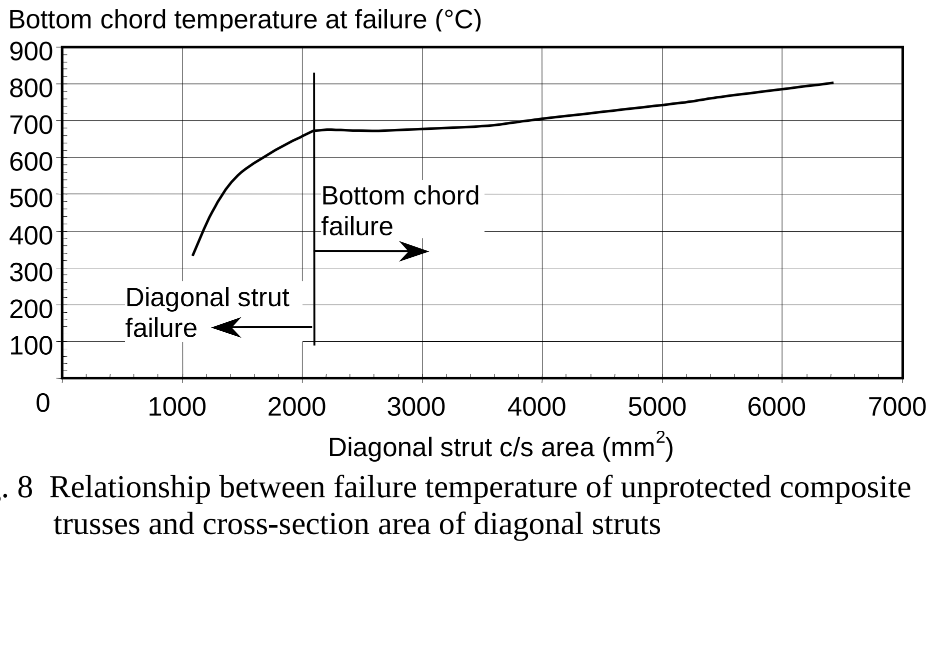


Fig. 8 Relationship between failure temperature of unprotected composite trusses and cross-section area of diagonal struts

It was found that the internal force of the first compressive diagonal strut increases by 45% when the member temperature reaches about 250^{o}C, due to the nonlinear thermal distribution across the slab and the difference of thermal elongation of steel and lightweight concrete. This phenomenon did not to induce any premature local instability into the models in this study.

4 PROTECTED COMPOSITE TRUSS SYSTEM SUBJECT TO THE ISO834 STANDARD FIRE

In order to achieve a fire resistance of 60 minutes in the standard fire, parts of the standard model were insulated by applying a thin-film intumescent coating and modified by using the knowledge obtained from the unprotected models.

4.1 Protected diagonal strut and bottom chord

Fig. 6 shows that all of the unprotected steel members reach above 900^{o}C in 60 minutes of the standard fire, at which point the strength retention of the material is less than 6%[3]. Figs. 2, 6 and 7 show that the standard model needs fire protection for the diagonal struts and bottom chord in order to achieve the required fire resistance period. Hence, all the diagonal struts and the bottom chord were assumed to be insulated. The temperature development of the protected members was programmed to increase rapidly to about 200°C in similar fashion to unprotected members, and then to progress linearly up to 620°C at 60 minutes in the standard fire (Fig. 9). This models fairly accurately the heating of members protected by intumescent paints.

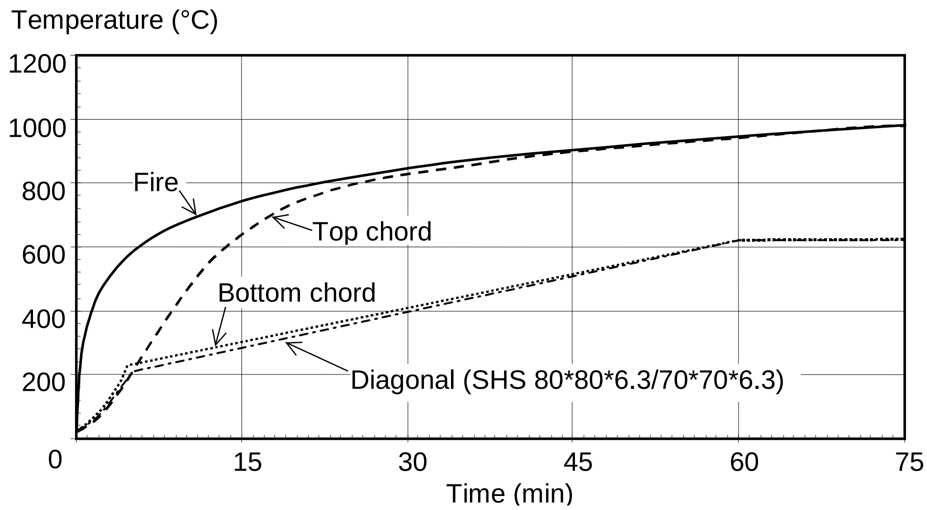


Fig. 9 Standard fire gas and steel temperature curves for unprotected top chord, with other members protected by intumescent paint

When examining ways in which to satisfy each member's conditions, such as material strength retention at the target temperature of the protected steel, internal force and the identified bucking pattern of pin-ended struts, three modified models were designed. These have SHS 70*70*6.3, SHS 80*80*6.3 and SHS 90*90*6.3 respectively as the first tension and compression struts, and the remaining strut members as SHS 70*70*6.3. Top and bottom chords were the same as the standard model in all cases.

The performance of the modified models in the standard fire is shown in Fig 10 in terms of the vertical deflection of the mid-span top chord in the period of the fire. It is shown that the model with the first pair of diagonal struts of SHS 70*70*6.3 loses stability in 55.3 minutes due to buckling of the first strut, and the other two models fail in 58.7minutes by yielding of the mid-span member of the bottom chord.

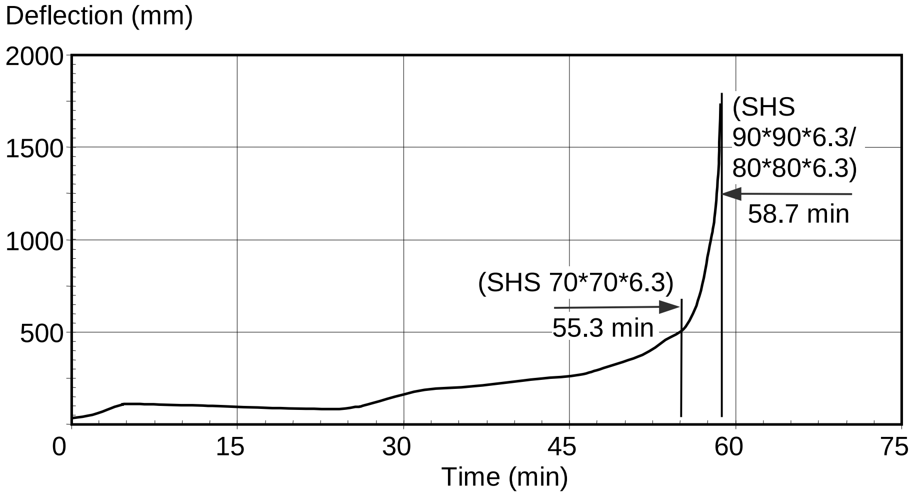


Fig. 10 Vertical deflection at the mid-span of the top chord of modified models in the standard fire.

4.2 Modified bottom chord

Fig. 10 shows that a modified model, with the first pair of diagonal struts made from SHS 80*80*6.3 and the remaining struts from SHS 70*70*6.3, is appropriate to avoid local instability of the diagonal strut. There is, however, still a need to increase the capacity of the bottom chord. Therefore, further numerical analyses were carried out with enlarged cross sections of bottom chord, as shown in Fig 11. The results show that the bottom chord needs to increase its area by 7.5% for the model to have 60 minutes resistance in the standard fire.

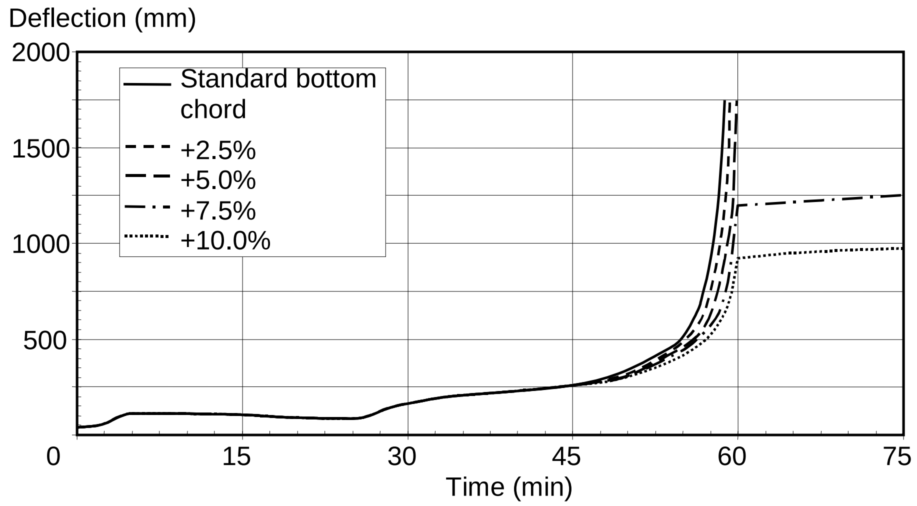


Fig. 11 Vertical deflection at the mid-span of the top chord of modified models with enlarged bottom chord in the standard fire

5 CONCLUSIONS

Numerical analysis using VULCAN, of a standard lightweight composite truss of 15.0m span at 3.0m spacing, designed as a simple composite beam without end-restraint for ambient temperature, shows that the model would fail at a uniform elevated temperature of 487°C. It was shown that, by increasing the diagonal strut from SHS 60*60*5 (1090mm^2) to SHS 90*90*6.3 (2090mm^2), the bottom chord temperatures at failure of unprotected trusses in the standard fire increase from 335°C to 670°C, which are equivalent to fire resistance times of about 7.0min to 15.0min respectively. In order to provide 60 minutes fire resistance in the standard fire, it was proposed that all of the diagonal struts and bottom chord should be insulated to stay below 620^{o}C at the target time period. The modified system, which has the first pair of diagonal struts of SHS 80*80*6.3, the remaining struts of SHS 70*70*6.3 and a bottom chord enlarged by 7.5%, meets the 60-minute requirement.

The pilot study forms part of a more general investigation aimed at establishing the key aspects of the performance of lightweight long-span floor systems in fire conditions.

Acknowledgment

The authors gratefully acknowledge the support of METSEC Building Products Ltd, and the collaboration of Buro Happold FEDRA.

REFERENCES

[1] *BS.5950: Part8: Code of Practice for the Fire Protection of Structural Steelwork*, British Standards Institution,, UK, (1990).

[2] *ENV 1993-1-2: Eurocode 3: Design of Steel Structures. Part1.2: General Rules: Structural Design for Fire,* Commission of the European Communities, Brussels (1993).

[3] *ENV 1994-1-2: Eurocode 4: Design of Composite Steel and Concrete Structures. Part 1.2: General Rules: Structural Fire Design*, Commission of the European Communities, Brussels (1994).

[4] *BS.5950: Part1: Code of Practice for Design: Structural Use of Steelwork in Building: Rolled and Welded Sections*, British Standards Institution, UK, (2000).

[5] Wainman, D. E. and Kirby, B. R., *Compendium of UK standard fire test data*, British Steel (Swinden Laboratory), (1987).

[6] Janss, J. and Minne, R., Buckling of steel columns in fire conditions, *Fire Safety Journal*, **4**, (1981/1982).

[7] Franssen, J. M., Janss, J. and Twilt, L, *The effect of the mechanical and structural properties of steel at elevated temperatures on the buckling of fire exposed columns*, ECSC Report TC3, (1989).

[8] Kirby, B.R. and Preston, R. R., High temperature properties of hot rolled structural steel for use in fire engineering studies, *Fire Safety Journal*, **13**, (1988) pp27-37.

[9] European Convention for Constructional Steelwork, *European recommendation for the fire safety of steel structures,* ECCS Technical Committee 3, (1981).

[10] Both, C. and van de Haar, P.W., *The mechanical behaviour of a fire-exposed two-span composite steel/concrete slab with Ribdeck 60 steel decking and specially draped mesh reinforcement: test report,* TNO-BOWUW report: 94-CVB-R1384, TNO Building and Construction Research, (1994).