Structural performance of long-span composite truss and beam systems in fire

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ABSTRACT

Composite truss and composite beam systems have recently been widely applied in long-span multi-storey floor construction. In a fire resistance design according to BS5950 Part 8, two alternative calculation approaches, a limiting temperature method and a moment capacity method, are used to assess the fire resistance of composite structures. The in-fire performance of the structures and insulation materials are required to satisfy a limiting strain level and a simple deflection criterion during the structure's specified fire resistance period. However, it is difficult to estimate the deflection of long span structures at elevated temperatures, due to the significant thermal effects on both the materials and structural mechanics. This project uses the Sheffield FE package '*Vulcan*' to investigate numerically the in-fire performance of two types of long span structure which are passively protected against fire.

INTRODUCTION

There have been significant improvements in the structural design of commercial multi-storey buildings in recent years, based on the development of long-span (12 to 20m) composite systems. The wide variety of automatically fabricated long-span composite structures has not only economic benefits due to fast, light-weight and accurate construction, but also allows the potential advantage of maximising flexibility in internal layout, which is one of the major trends in modern commercial buildings. The configuration of a composite truss, which generates large voids between the chord and bracing members, provides that the services can pass at any location through the floor zone without modification. A composite cellular beam may also offer a similar advantage by using its web openings. With regard to the optimum integration of structure and services within the same horizontal zone, composite beams and composite trusses are two of the most common structural systems which have been widely used for long span floors.

In a typical design procedure, after a composite structure has been designed for ultimate limit state, its design for the fire limit state is subsequently done for the specified fire resistance period. Two fire safety design philosophies, prescriptive and performance-based, may be used to achieve the required fire resistance against standard time-temperature curves, such as “the Standard Fire” (ASTM E119 (ASTM 1995), BS476 Part20 (BSI 1987) or ISO 834 (IOS 1975)), or the parametric fire of Eurocode 1 Part1.2 (CEN 2001).

In prescriptive design manufacturers’ data is used to estimate a protection thickness based solely on keeping the structure temperatures below certain fixed values (550°C is the most usual of these). In the performance-based design philosophy, the behaviour of the structures under fire may be presented as a specified deflection against time using numerical analysis. More simply, for individual members, both BS5950 Part 8 (BSI 1990) and Eurocode 3 Part 1.2 (CEN 1993) provide “critical temperature” methods based on previous testing, in which the critical temperature is a function of the member’s load ratio. The fire performance of such members may be improved by varying the section size, the protection material thickness etc. The fire resistance of the members may also be calculated using the moment capacity method in accordance with Eurocode 4 Part1.2 (CEN 1994). In the UK, the fire performance of composite members and protection materials are typically specified up to a 2.0% maximum strain level according to BS5950 Part 8 (BSI 1990) and a deflection limit of span/20 is commonly used in structural fire design practice. However, due to the significant thermal effects on the material properties and the structural mechanics of load-carrying, it is difficult to calculate the deflection of such long-span composite systems at elevated temperatures.

The current understanding of structural behaviour in fire reaches beyond individual member performance. Each member contributes to the overall performance of a structural assembly in fire, but this is also affected by their interaction in a truly performance-based design method. The purpose of this study is to investigate numerically the performance of a composite truss with a supporting column, for 60 minutes of the standard ISO fire. This is compared against the fire performance of equivalent composite beams, which are protected using the prescriptive method.

MODELLING

A standard model of a Warren Truss, of 15m span and 750mm depth, spaced at 2.5m centres and made composite with a concrete slab and profiled steel decking, has been used for this study (Figure 1). The top and bottom chords were adapted from a unique S350 METSEC cap section (300mm wide × 100mm deep × 8mm thick) and the web members from RHS60 × 60 × 5.0. The model has been designed to satisfy ultimate strength and deflection criteria at an ultimate limit state loading of 13.2kN/m^2, according to BS5950 Part1 (BSI 2000). The steel decking 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. Imposed loading at the fire limit state is 7.81kN/m^2, which is a typical office floor loading factored according to BS5950 Part 8 (BSI 1990), for this composite truss.

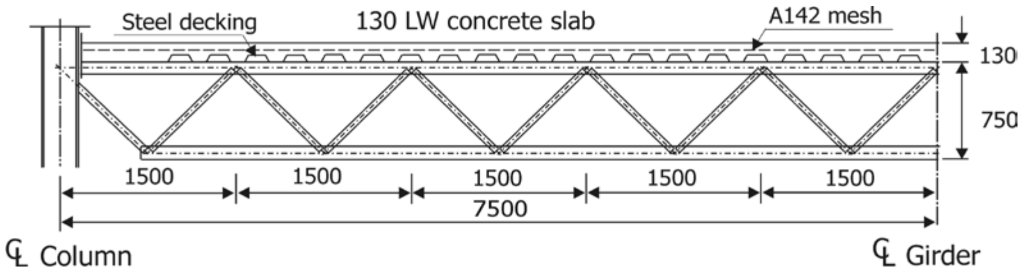

Figure 1 Half span of the composite truss (dimensions in mm)

Two composite beams, shown in Figure 2, were designed according to BS5950 Part1 (BSI 2000), for conditions identical to those which applied to the composite truss. The cellular beam of Type A, which had a 150mm × 10mm top flange, 220mm × 15mm bottom flange, 8mm thick web and 400mm diameter web-openings at 600mm centres, was assumed to be fabricated from welded S355 steel plates. The Type B model also contained 450mm diameter of web-openings, in this case at 650mm centres, and was constructed from two standard universal beams (S355) cut along their webs. It had dimensions of 152.4mm × 10.9mm flange and 7.6mm thick web top tee; 190.4mm × 14.5mm bottom flange and 9.0mm thick web bottom tee.

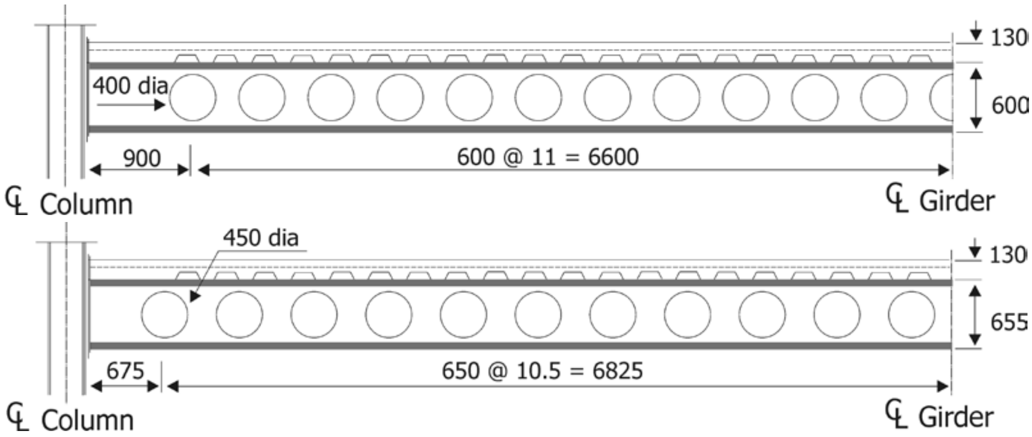

Figure 2 Half span of Type A (above) and B (below) composite beams (dimensions in mm)

For this study, a symmetric half of the composite truss was numerically modelled with a 4m supporting column (UC 356 × 368 × 129). A pinned beam-to-column connection was assumed between the steel members. The proposed model was designed assuming a continuous bottom chord and top chord with the LWC slab, excluding the steel decking from the model. The pin-ended web members were connected between the centroids of the top and bottom chords. All transverse loading was applied at the connections of the top chord and bracing members. No eccentricity was assumed at the connections. For the numerical modelling of the composite cellular beams, the circular openings of diameter *d* were transformed to be effective rectangular openings of depth *d* and width *d/2*.

TEMPERATURE ESTIMATION

In order to achieve a fire resistance of 60 minutes in the standard fire, the web members and the bottom chord of the composite truss, and the whole surface area of the composite cellular beams, were assumed to be insulated by applying a thin-film intumescent coating. The design temperature of the protected steel was considered to be 620°C. The temperature development of the protected members was assumed to increase to about 200°C in a similar fashion to the unprotected members. The temperature development at this stage was calculated on the basis of the section factor, *A/V*, using a formula provided by Eurocode 4 Part1.2 (CEN 1994). The temperature increase, $\Delta\theta_a$, of unprotected steel chords during a short time interval, Δt, may be determined by following formula:

$$\Delta\theta_a = \frac{\alpha_c + \alpha_r}{C_a \rho_a} \cdot \frac{A}{V} \cdot \left(\theta_t - \theta_a\right) \cdot \Delta t \ \ [°\mathrm{C}] \qquad [1]$$

where α_c is the coefficient of convective heat transfer

α_r is the coefficient of radiative heat transfer

$$\left[= \Phi\left(\frac{5.67\times10^{-8}\,\varepsilon_{res}}{\theta_t - \theta_a}\right)\left(\left(\theta_t + 273\right)^4 - \left(\theta_a + 273\right)^4\right)\right]$$

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 (conservatively taken as 0.5 for steel and 0.56 for concrete)

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 part of the steel cross section per unit length (m^2/m)

V is the volume of the part 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)

Δt is time interval (sec)

After this the temperature was assumed to progress linearly up to 620°C at 60 minutes for beams and 550°C for the supporting column, as shown in Figure 3.

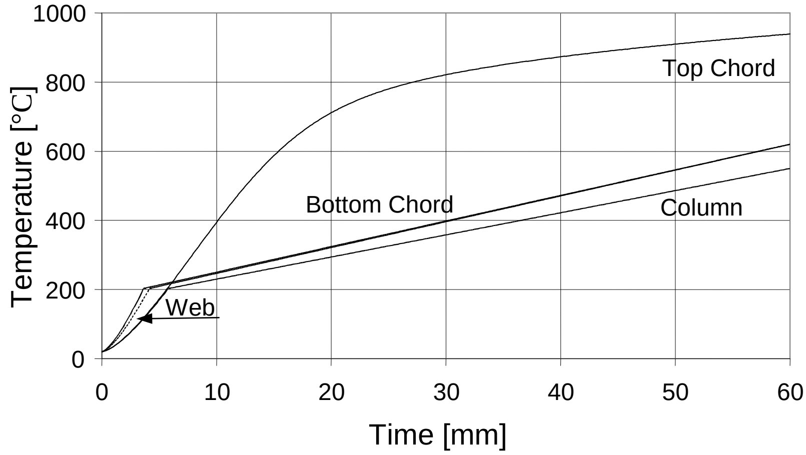

Figure 3 Temperature development of composite truss

The temperature profiles of the cellular beams were calculated in the same manner for the top flange, web and bottom flange parts of the section respectively, using their individual section factors. BS5950 Part8 (BSI 1990) and *Fire Protection for Structural Steel in Buildings* (ASFP 2002) recommend that the protection thickness of the cellular beams should be obtained based on the original cross-section and then increased by 20%, as the temperature of the cellular beams increases at a faster rate than conventional sections.

A thermal analysis was conducted, using heat transfer software (Huang *et al.* 1996), to generate the temperature development of the 70mm LWC slab topping, divided into 9 layers with the steel decking, in the standard fire (Figure 4). The moisture content was taken to be 2% of the concrete weight, and coefficients of heat transfer were chosen from generic data (Purkiss 1996).

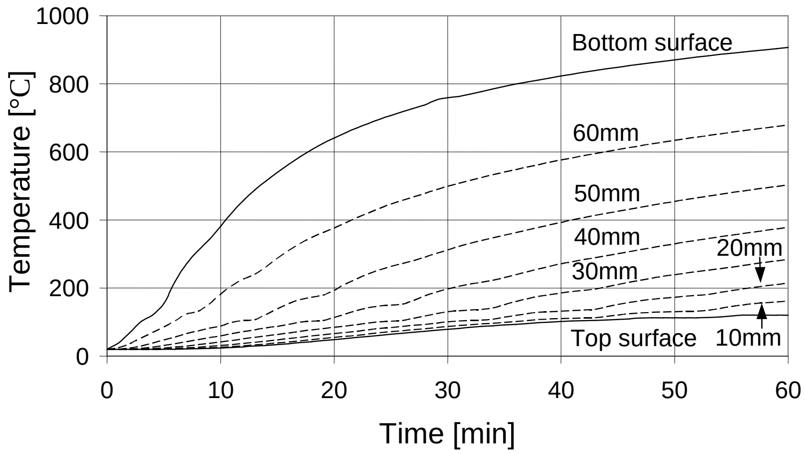

Figure 4 Temperature development of 70mm topping of the LWC slab

PARAMETRIC STUDY

Numerical analyses of the structural behaviour of the composite truss and composite beams, subjected to 60 minutes exposure to the ISO834 standard fire, were conducted using the finite element program *Vulcan*, which has been specifically developed by the Structural Fire Engineering Group at the University of Sheffield in order to investigate structural performance at elevated temperatures.

The performance of concrete and steel at elevated temperatures varies inelastically with their temperature-dependent characteristics such as strength, stiffness, thermal expansion, specific heat and thermal conductivity. For numerical analysis the mechanical and thermal characteristics of these materials were formulated in accordance with Eurocode 3 Part 1.2 (CEN 1993) and Eurocode 4 Part 1.2 (CEN 1994). The tensile strength of the concrete at elevated temperatures was also taken into account by adapting a tensile stress-strain curve. The curve was modelled to have linear behaviour up to the peak tensile strength $f_t(\theta)=0.3321\sqrt{f_c(\theta)}$, and then a bilinear curve (Rots *et al.*1984) for tensile strain-softening after cracking, up to the maximum tensile strain $\varepsilon_{ct}(\theta)=15f_t(\theta)/E_c(\theta)$. After tensile cracking the concrete was still capable of carrying compressive stress, but was ignored after crushing in compression.

Composite Truss

The normal equilibrium of one symmetric half of the composite truss, under simply supported conditions at ambient temperature, is illustrated in Figure 5. Double black- and white-filled arrowheads indicate compression and tension respectively. Single black vertical arrows represent resultant shear forces. The critical elements of this 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 members in compression the critical member is the first compressive diagonal strut at 0.75–1.5m from the support. As shown in a similar model study in previous publication (Choi *et al.*, 2002), these may also be the critical elements under elevated-temperature conditions.

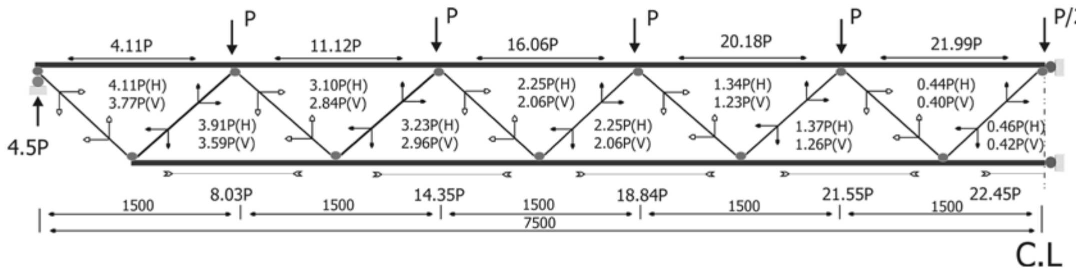

Figure 5 Equilibrium of the composite truss at ambient temperature

It was observed that the first and the second compressive web diagonals may buckle, causing a local instability within the fire resistance period. Once this condition occurs, the moment resistance of the composite truss, generated by the lever arm between top and bottom chords, ceases to be valid. The load-carrying mechanism of the composite truss is transformed, so that the slab and top chord both begin to carry the imposed load in tension rather than in compression. This phenomenon is called 'catenary action', and relies on the truss undergoing large deflection in the process. In this situation the top chord and slab, with an embedded 25mm reinforcement bar, may possibly be able to survive during the fire resistance period, although with large deflections. In order to reduce the severity of this deflection, web instabilities were avoided by adopting RHS100 ×6 0 × 6.3 for the web diagonal members. With this modification the composite truss maintained a flexural mechanism during the fire resistance period and deflected by approximately *span/20* (750mm) at 60 minutes of the Standard Fire, as shown in Figure 6. The strain level in the bottom chord remained within 2.0% during the fire resistance period.

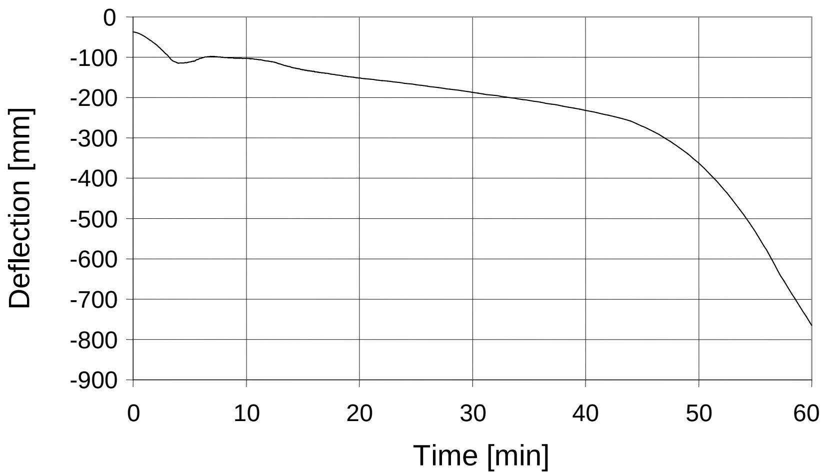

Figure 6 Mid-span deflection of the composite truss

Composite Cellular Beam

The structural behaviour of Type A and B composite beams, with and without openings, for 60 minutes of the standard ISO fire, was numerically investigated using *Vulcan*. The mid-span deflection of each of the models is plotted against time in Figure 7. In the cases of the composite beams without web-openings, which are represented by solid lines, Type B shows the better performance, due to its increased beam depth. When openings are included (dotted lines), Type A deflects less because of its larger bottom flange area, smaller ratio of opening to overall beam depth and longer distance from the first opening to the support.

The load ratios of the composite beams, with and without openings, were 0.52/0.39 for Type A and 0.53/0.38 for Type B respectively. With respect to both of the prescriptive methods, the protection temperature of 620°C was conservative for the composite beams. Therefore, the models, with and without web openings, deflected by approximately *span/15* and *span/20* during the fire resistance period.

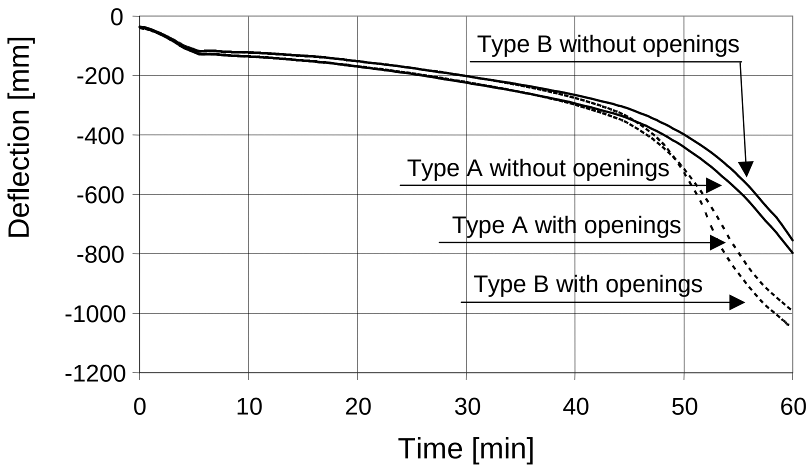

Figure 7 Mid-span deflections of the composite beams

CONCLUSIONS

The structural behaviour of composite beams and composite trusses, as alternative long-span (15m) floor systems at 2.5m spacing, for 60 minutes of the standard ISO834 fire, was numerically investigated using *Vulcan*. It was shown that the composite truss (750mm depth), designed at ULS and protected using the prescriptive method, may need further consideration in design for fire, specifically to avoid web instability, which would induce a sudden transition to a large-deflection load-carrying mechanism. The numerical study shows that the composite truss, protected to reach steel temperatures no higher than 620°C under a load ratio of 0.5 at the fire limit state, deflects by approximately *span/20* during the fire resistance period. The equivalent Type A (600mm depth) and Type B (655mm depth) composite beams with and without web openings, which were also protected to 620°C under load ratios of 0.52/0.39 & 0.53/0.38, were demonstrated to deflect by approximately *span/15* and *span/2*0 during the fire resistance period. All of the cases satisfied the limiting strain criterion of 2.0% throughout the analyses. However, despite the conservative protection temperature under the load ratios considered in the limiting temperature method, the results show that the deflection limit of *span/20* is difficult to achieve from either long span composite beams or trusses in fire.

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