# Development of a light-weight composite lattice joist for fire resistance

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ABSTRACT: This paper describes development studies on a new composite lattice joist floor framing system, particularly on the performance-based approach taken to fire resistance. This has involved collaboration between the manufacturer, a consulting engineer and university researchers. The light-weight open-web floor joist is fire-protected offsite using intumescent paint, to achieve a one-hour fire rating. It is a competitive light-weight option for long-span floor construction, which allows ducting to pass through the voids in the lattice. It eliminates the need for through-deck welding on site due to the presence of pre-welded shear connectors in the top chord. The main objective was to develop a cost-effective method of achieving one-hour fire resistance using a single coat of intumescent paint. This system is common in hot -rolled steel construction, but has not usually been associated with light-weight cold-formed sections. The system has been analysed using the specialized software *Vulcan*. The prototype design was based on 750mm deep Warren joists of 15m span, spaced 2.5m apart, with a composite concrete slab on profiled steel decking. A BS476 loaded fire test was carried out at the Warrington Fire Research Centre which verified the software approach. Recent modelling has focused on developing a series of such joists to be used over a range of spans.

## 1 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 economic benefits, due to fast, light-weight and accurate construction, and also allows the potential advantage of maximising the flexibility of the internal layout, a major trend in the design of modern commercial buildings. The configuration of a composite truss, which contains large voids between the chord and bracing members, allows services to pass at any location through the floor zone without modification. A composite cellular beam may also offer a similar advantage through the use of its web openings. This optimum integration of structure and services within the same horizontal zone means that composite beams and composite trusses have become two of the most commonly used structural systems in buildings with long-span floors.

The market size for cellular beam has significantly increased in the UK in recent years to more than 30,000 tonnes per year. However, the composite truss, which is already used widely in the USA, is not yet common in the UK. One of the obstructions to the adoption of a composite truss system in the modern office building market of the UK is the fire resistance design. Unlike composite solid-web beams, which have been thoroughly tested at elevated temperatures and possess well established fire design methods, the in-fire performance of the composite truss system has not yet been fully investigated. As the composite truss is composed of top and bottom chords and a series of web members, it acts as a combination of load-bearing elements; hence, typical insulation methods for composite beams may not be appropriate for composite trusses. Furthermore, should a situation arise where the understanding of structural behaviour in fire reaches beyond individual member performance, each member's contribution to the overall performance of a structural assembly in fire will require examination. In addition, since this type of structural system was used for the floors of the collapsed twin towers of the World Trade Center, a detailed investigation of the system in fire has been also demanded by the public.

## 2 DESIGN CRITERIA

Composite trusses become cost-competitive for spans greater than 12m, and present the most economic structural system when spans are 18m or longer. The most common truss spacing is 3.0m, with the maximum being approximately 3.5m. The exact spacing tends to be determined by the capacity of single spans of the corrugated steel decking when it is required to support wet concrete during placement. Typically, the composite trusses form one-way secondary framing elements and support the decking directly.

After the composite structure has been designed for ultimate limit state, its design for the fire limit state is carried out for a specified fire resistance period. Two alternative fire safety design philosophies, prescriptive and performance-based, may be used to achieve the required fire resistance, both usually under the standard time-temperature curve defined in Eurocode 1 Part 1.2 (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 performance-based design, the behaviour of the structure in fire may be presented as a specified deflection against time, found using numerical analysis. For individual members, both BS5950 Part 8 (1990) and Eurocode 3 Part 1.2 (2005) provide "critical temperature" methods based upon 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 Part 1.2 (2005). 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 and a deflection limit of span/20 is commonly used in structural fire design practice. However, due to the significant effects of temperature on 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.

In a general design process for composite trusses at the ultimate limit state, the chord and web members of the system need to exceed their total factored applied moments and shear forces respectively. The flexural capacity of a composite truss can be estimated from the couple between the slab in compression and the bottom chord in tension. In this calculation, the required section size of the bottom chord may be assessed, but the contribution of the top chord is generally ignored as it lies very close to the neutral axis of the composite truss. Web elements are designed to carry all shear forces and, for in-plane buckling of the compression diagonals, the effective length is taken as 0.85 times their system length between nodes.

In order to achieve a ductile failure mechanism in the composite truss, the ultimate tensile force of the bottom chord needs to be adjusted so that it is exceeded before either the concrete slab or web members fail in tension or compression. This failure pattern is also the preferred mechanism at the fire limit state.

No eccentricity is assumed at the joints between chords and webs. The top chord needs to be designed under construction loads, and during the composite stage to satisfy the base-metal thickness requirement for stud welding. At the serviceability limit state, deflection criteria must be checked at both the non-composite and composite stages of construction, as well as vibration serviceability.

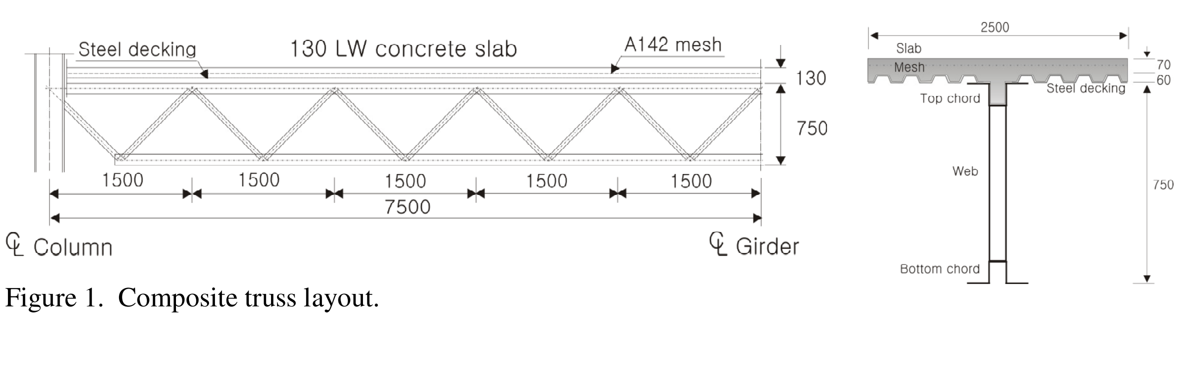


Figure 1. Composite truss layout.

In this study a model Warren truss with 15.0m span and at 2.5m spacing, composite with a concrete slab and profiled steel decking (Figure 1) was used. Ultimate limit state loading was calculated according to BS5950 Part 1, using a characteristic dead load of 3.6kN/m$^2$ and a live load of 5.0kN/m$^2$. The system was assumed to be one-way acting and to possess simple support conditions. The depth of the truss, 750mm, was calculated using the typical span/depth ratio (span/20) for a simply supported beam. The slab loading, distributed along the member length, was converted into a series of point loads, positioned at the truss joints between the top chord and web members. All the truss joints were assumed to be pin-connected. Full shear connection between the top chord and concrete slab was also assumed. The chord members were specified as being:

- A cold-formed "cap" section (300 mm width, 100 mm depth and 8 mm thickness), produced by Metsec Building Products Ltd, which has a yield strength grade of 350N/mm$^2$. The top chord was assumed to be concrete in-filled to avoid local instability.
- The steel decking (Z35: 350N/mm$^2$ yield strength) was presumed to act as permanent formwork to the underside of the light-weight concrete slab.
- The slab itself, which has 30N/mm$^2$ cube strength. A142 (142mm$^2$ per metre width) steel mesh reinforcement was placed in the concrete slab for cracking resistance during curing.
- RHS 60x60x5 (355N/mm$^2$) hollow sections were used for web diagonals in the initial model, but larger sections such as RHS 90x50x5, RHS 100x50x5, RHS 100x60x5 and RHS 100x60x6.3, were subsequently used in order to improve the in-fire performance of the composite truss.

This proposed model was inherently over-designed (possessing a moment ratio of 0.87 at ultimate limit state) to optimise its performance in fire in relation to various protection scenarios and boundary conditions.

## 3 NUMERICAL MODELLING

The temperature development of each member in the steel joist and each segment of the concrete slab was assessed for unprotected and protected conditions, during 60 minutes of a Standard Fire. This considered the economics of member size and insulation material required.

### *3.1 Geometry and material modelling*

In the numerical model, the geometry of the composite truss was assumed to be such that there was no eccentricity at the connections. The origin was arbitrarily located at the left end of the top chord, with the z-axis directed longitudinally and the y-axis in the vertical plane of the truss (positive upwards). The horizontal x-axis was perpendicular (out of plane) to the truss. The truss was assumed to be simply supported at its top chord and perfectly straight. In such a composite truss, the member behaviour is dominated by axial forces, which are usually determined by pin-jointed truss analysis. The proposed model was more realistic, using a continuous concrete slab and top chord, with node points positioned at the centroid of the top chord and a continuous bottom chord. A series of pin-ended diagonal web elements link the chords. The proposed model provides a better representation of overall structural performance, yet preserves the conservative assumptions concerning the web members, which were modelled as being pin-ended but are, in fact, welded to the chords during construction.

Like other composite structures at elevated temperatures, the performance of the model composite truss was likely to be significantly influenced by its material characteristics, such as strength, deformation, thermal expansion, specific heat and thermal conductivity – all being temperature-dependent (the high-temperature performance of materials being dependent on their chemical composition and atomic structure). The moisture content of some materials, such as concrete, also required inclusion in order to predict their characteristics at elevated temperatures. The mechanical and thermal properties of steel and light-weight concrete, which are the two materials used in the composite truss, at elevated temperatures, were numerically formulated according to Eurocode 3 Part 1.2 and Eurocode 4 Part 1.2. The tensile strength of light-weight concrete, which is not included in Eurocode 4 Part 1.2, was taken into account by adopting the series of stress-strain curves proposed by Rots *et al.* (1984) and tested by Huang & Platten (1997) and Cai *et al.* (2003). The stress-strain curve was modelled as linear up to the peak tensile strength, $f_t(\theta)=0.3321\sqrt{f_c(\theta)}$, and does not exceed 10% of the corresponding compressive strength suggested by Eurocode 4 Part 1.2. Beyond this strain, a bilinear curve was assumed for tensile strain-softening after cracking (Barzegar-Jamshidi 1987), up to the maximum tensile strain $\varepsilon_{ct}(\theta)=15f_t(\theta)/E_c(\theta)$. After cracking under tension the concrete was still able to carry compression, but after crushing in compression all strength was assumed to be lost.

### *3.2 Temperature assessments of steel members and concrete slab*

In order to conduct a finite element (FE) study of the composite truss under the Standard Fire, a preliminary examination of the temperature development in the steel members and concrete slab is typically required. The time-temperature relationships of unprotected steel members may be assessed on the basis of their section factors, using a formula proposed in Eurocode 4 Part 1.2. In this model, as the practical range of the unprotected steel members tended to reach over 600°C within 15 minutes of the onset of the Standard Fire, an off-site intumescent coating was suggested that would insulate the steel members to survive for 60 minutes of the fire. The temperature profile of the light-weight concrete slab was obtained using a heat transfer package FPRCBC-T (Huang *et al.* 1996).

The temperature development of unprotected top and bottom chords and two representative web members (A/V=215m$^{-1}$ and 170m$^{-1}$) under the Standard Fire were calculated. The chosen A/V values can represent sections of RHS 60x60x5 (A/V=218m$^{-1}$), RHS 90x50x5 (A/V=215m$^{-1}$), RHS 100x50x5 (A/V=214m$^{-1}$), RHS 100x60x5 (A/V=213m$^{-1}$), and RHS 100x60x6.3 (A/V=170m$^{-1}$), The bottom chord reaches temperatures above 700°C within 15 minutes of the start of the Standard Fire, when 23% of the steel strength remains. The web members reach 800°C at 30 minutes of the fire, when 11% of the steel strength remains. It is obvious that the majority of the load-bearing steel members may require insulation to resist 60 minutes of the Standard Fire. An off-site intumescent coating was chosen as the fire protection method to insulate the steel members to survive for 60 minutes. The temperature of an insulated steel member increases rapidly to about 200°C, as for the unprotected condition, and then progresses linearly up to the target temperature at the end of the fire resistance period. The required coating thicknesses of protected hollow-section web and bottom chord members were proposed by Nullifire Ltd on the basis of experimental data. Target member temperatures were intended to be achieved using maximum single or double coating thicknesses. In the case of web members with A/V=215m$^{-1}$ and 170m$^{-1}$, using a single coat the temperature of the steel reaches 750°C and 650°C respectively at 60 minutes. The bottom chord should reach similar target temperatures, whilst the top chord was unprotected. As intumescent coatings are comparable in price to steel, it was intended to maintain a single coat for both web and chord members.

As the concrete slab carries the compressive force in pure bending, it was essential to identify the temperature profile through the depth of the slab. This was obtained numerically for a light-weight concrete slab at defined time-steps, using FPRCBC-T.

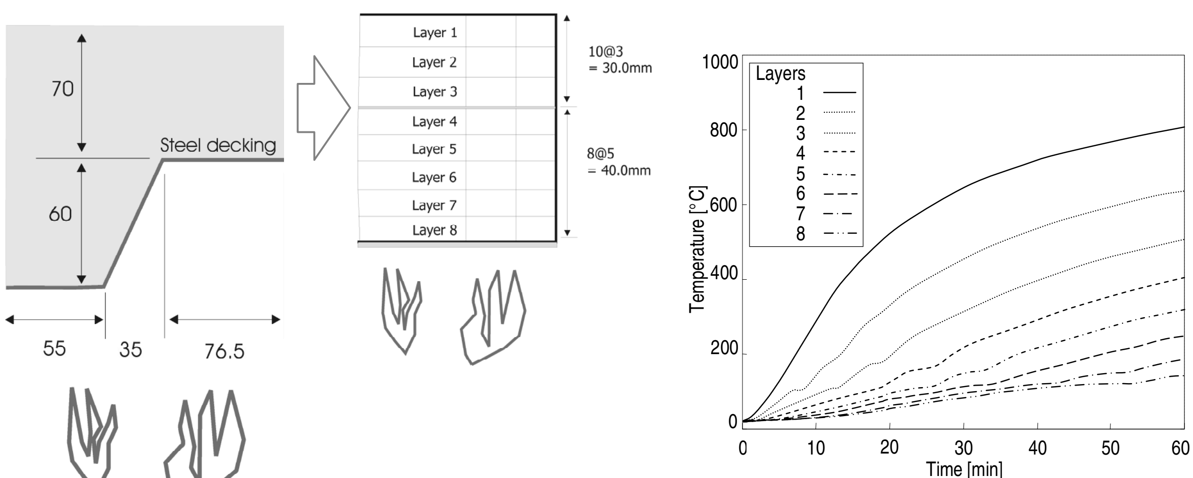


Figure 2. Thermal analysis of concrete slab.

Thermal analyses were conducted for an equivalent flat slab including 4% moisture content. Coefficients of heat transfer were conservatively chosen from generic data given by Purkiss (1996): View factor = 1.0, flame emissivity = 1.0, surface emissivity = 0.8, surface absorptivities = 0.625 (steel) and 0.7 (concrete), convection factors = 2.2 (fire-exposed boundary) and 1.0 (ambient-temperature boundary), convection powers = 1.25 (fire-exposed boundary) and 1.33 (ambient temperature boundary). The 70mm topping of the slab was modelled in 8 layers, with an anti-crack mesh and a steel decking, as shown in Figure 2. The temperature development at the mid-point of each layer of the topping is also shown. It should be noted that the slab, from the top surface to 30mm depth, remains below 300°C throughout 60 minutes of the Standard Fire, so that the ambient-temperature strength of the concrete is retained, indicating that the compression block maintains its strength, so that its depth is reduced, during the fire resistance period.

## 4 UNRESTRAINED COMPOSITE TRUSS

The behaviour of unrestrained composite trusses, designed optimally for ambient temperature, was investigated using the ISO834 Standard Fire and *Vulcan*. The contribution of top and bottom chords and web members to the overall performance was examined to optimise the section-sizes of the members with respect to intumescent protection, in order to achieve a 60-minute fire resistance period. A 15m example is illustrated here.

### *4.1 Unprotected condition*

The equilibrium of the composite truss before any temperature rise was analysed using *Vulcan*. The highest value of load ratio in tension was that of the mid-span bottom chord, and in compression in the compressive web members nearest the supports. These are also typically the critical elements at elevated temperature. Partial safety factors for loads at the Ultimate and Fire Limit States differ, and the actual stress in individual members at the Fire Limit State is typically less than 60% of their ultimate strength. In this study the Fire Limit State loading of 7.81kN/m$^2$, assessed according to BS5950 Part 8, delivers a moment ratio of 0.50 to the model. A series of analyses was carried out on various unprotected composite truss models at the Fire Limit State. The same basic arrangement and member specifications were used in all cases, with only the cross-sectional areas of the web members being altered between analyses. Typical results are shown in Figure 3 in terms of deflection against both time and the temperature of the bottom chord. It can be clearly seen that, through modification of the area of the web member, the fire resistance of the unprotected truss can be improved so that the bottom chord achieves a temperature of 664°C before runaway.

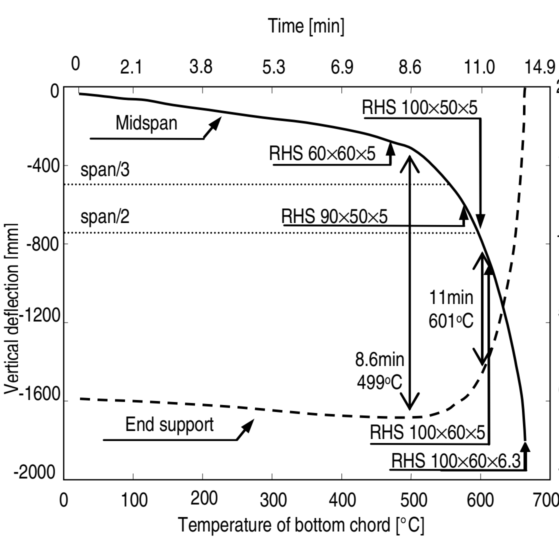


Figure 3. Deflections of unprotected truss.

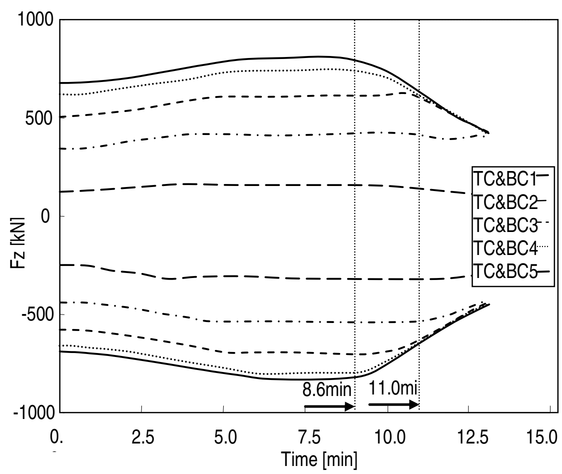


Figure 4. Top and bottom chord forces.

Since the stiffness of a composite truss is mainly determined by the flexural couple between the top chord plus slab and the bottom chord, the variation of the internal forces of the chords against time is shown in Figure 4 (‘TC #’ and ‘BC #’ are used to identify the chords members, and are numbered from the support towards mid-span). It can be seen that, over the first 8 minutes, differential thermal elongation causes the member forces to increase by 20 to 27% in compression and 20 to 34% in tension from the mid-span towards the end support. However, a noticeable decrease occurs from 8.6 minutes onwards.

It is possible to estimate the sagging moment of resistance of the composite truss in fire by considering the strength reduction of the bottom chord only, as the strength reduction of the compressive block in the concrete slab is relatively small and tensile resistance is mainly proportional to the strength reduction of the material at elevated temperatures. The load ratios in flexure ($LR_f$) and pure tension ($LR_t$) are also based on the strength reduction factor ($SRF_{2.0}$) at 2% strain, as follows:

$$LR_f = \frac{F}{A_g p_y SRF_{2.0}} + \frac{M_z}{M_{cz} SRF_{2.0}} + \frac{M_y}{M_{cy} SRF_{2.0}} \quad (1)$$

$$LR_t = \frac{F}{A_g p_y SRF_{2.0}} \quad (2)$$

Based on Equations (1) and (2), an evaluation of load ratio of the mid-span bottom chord indicates that, when the time reaches 8.6 minutes and 11.0 minutes respectively, the critical member achieves load ratios of 1.0 in flexure and pure tension. The change of load ratio of the critical bottom chord suggests that thermal elongation causes an increase in internal force up to the point at which the limiting flexural capacity is reached. Above this point, the deflection rate of the entire composite truss increases considerably and the direction of movement in the end supports changes from outward to inward. This is illustrated in Figure 3. After the full tensile capacity of the critical section is reached, the deflection of the system adopts a 'run-away' pattern. The reference deflections of span/30 and span/20 are observed to occur between these critical points.

In order to achieve the preferred flexural load-carrying mechanism in the composite truss during the fire resistance period, premature local instability of the web members must be avoided. Figure 5 demonstrates that the axial forces caused by thermal expansion in the web members increase during the early stages of the fire. On this figure 'CW#' and TW#' denote compressive and tensile web members respectively. These are numbered from the supports towards mid-span. As the axial forces in the top and bottom chords began to decrease from the mid-span to the end-support members (Figure 4), the corresponding forces of the web members was monitored. Of particular interest were those within the high-shear zone (within a distance up to twice the truss depth from the end supports), which were observed to undergo a considerable increase in axial force. After 2.5minutes, the axial forces had increased by 20% and, within 9.4minutes this had risen to 35%. Members between twice to four times the depth from supports also experienced a rise of about 18% in axial force.

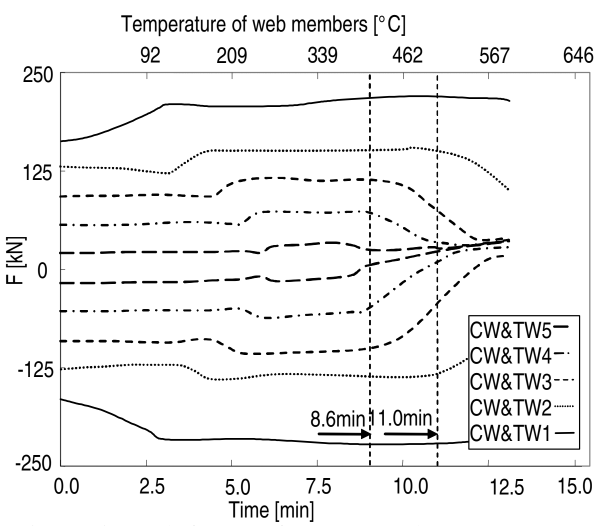


Figure 5. Web forces of unprotected truss.

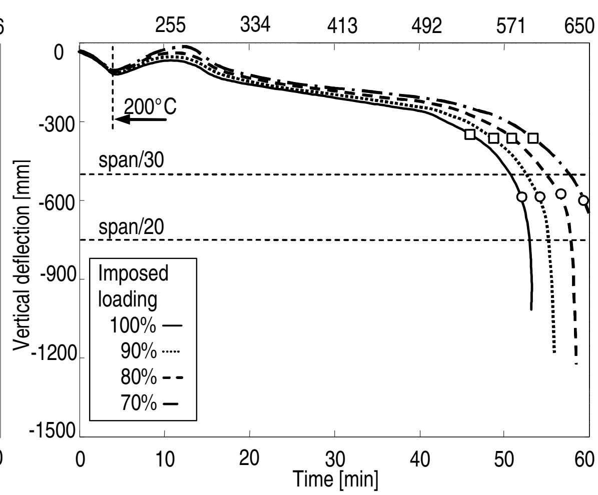


Figure 6. Deflections at different loadings.

Although the members in all cases were deliberately over-designed for ULS, thermally induced stresses, in addition to material strength reductions at high temperature, were observed to instigate premature local instabilities of the first and the second pairs of compressive web members. Failure times for the first compressive web members were found to be 8.1 minutes (RHS 60x60x5), 10.4

minutes (RHS 90x50x5), 11.0 minutes (RHS 100x50x5) and 11.4 minutes (RHS 100x60x5). Other web members were seen to acquire insignificant thermal stresses, but the lack of stiffness provided by the simply-supported end conditions, and subsequent redistribution of load following buckling of the first compression member, were sufficient to cause the system to collapse. In order to avoid such a web buckling failure, it was decided that the first pair and second compressive web members should be increased to RHS 100x60x6.3. Other web members were changed to RHS 60x60x5.0 for practical construction reasons, so that only two RHS sizes were used for the web elements.
For the unrestrained model to meet the span/20 deflection criterion, the bottom chord should be maintained at a temperature below 595°C. From Figure 5, it is evident that a web member of RHS 100x60x6.3 will tolerate the increased internal stresses caused by heating of up to 591°C.

### *4.2 Protected condition*

Although the failure mode of the truss can be altered solely through member modifications, a partial application of insulation was still necessary in order to achieve 60-minute fire resistance. From the member load ratios and the necessary material strength retention at the design temperature, insulated member temperatures were generated, and a new configuration of web members was generated. For the first and second pairs, RHS 100x60x6.3 were specified, whilst the other members were RHS 100x60x5.0. The bottom chord elements were assigned a maximum design temperature of 650°C (two layers of intumescent coating), and the RHS 100x60x6.3 and RHS 100x60x5.0 web members were assigned 650°C and 750°C respectively (one layer of intumescent). The top chord remained unprotected, as it does not affect the in-fire performance. The performance of the insulated composite truss systems was then numerically investigated. The system was subjected to a range of 100% to 70% FLS loading of 7.81kN/mm$^2$ (equivalent moment ratios of 0.50 to 0.35) and the deflection performance of the model was obtained using *Vulcan* (Figure 6). After approximately 4 minutes of the Standard Fire, the temperature of the bottom chord rises above 200°C and the insulation material activates. During the subsequent 10 minute period, an upwards deflection (reverse thermal bowing) is seen to occur. This results from the significant difference in temperature development between the compressive and tensile blocks within the system. After this period the deflection gradually increases in line with the rise in temperature of the bottom chord. The square and round boxes in Figure 6 indicate the points where the mid-span bottom chord reaches its full flexural and tensile capacities according to Equations (1) and (2). When the full flexural capacity is reached, the mid-span deflection is approximately span/43 .From this point onwards the rate of deflection increases rapidly until the tensile capacity is achieved at a deflection of span/25.

## 5 CONCLUSIONS

The work presented here has primarily been concerned with increasing understanding of the structural behaviour of long-span composite trusses in fire. The knowledge gained can be used to develop optimal fire designs for composite truss systems using performance-based design methods. With respect to the critical elements, two failure mechanisms in fire were identified:

- *Web member buckling failure*: This mode results in a sudden loss of stiffness, and possibly of load-carrying capacity.
- *Mid-span yielding of the bottom chord*: This mode results in a ductile deflection of the truss.

It was also shown that the model optimally designed for ULS may have a higher chance of experiencing a web buckling failure in fire due to the localised effects of thermal stressing. Following cross-section modification of the critical web members via the partial application of insulation materials and the use of tensile reinforcing bars, the performance of a restrained model (under a load ratio of 0.5) was enhanced sufficiently to sustain 60 minutes of the Standard Fire whilst deflecting by 1400mm.

### *5.1 The outcome of the work*

The studies described in this paper constituted an initial stage in the development by Metsec Ltd in collaboration with the structural engineers Buro Happold Ltd and the University of Sheffield of a new light-weight flooring system designed to permit the integration of service ducts within the depth of the flooring support girders. The new system's usage is illustrated in Figure 7. This is very much in tune with current requirements to fabricate and fire-protect large components off-site, to integrate building services and structure, and to maximize clear spans for open-plan office layouts.

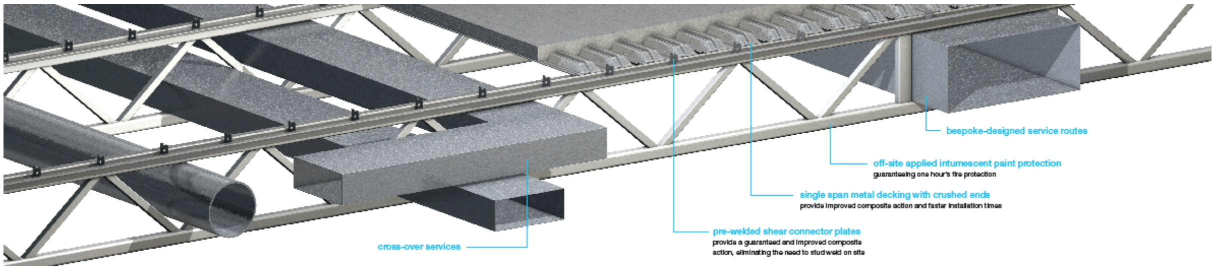


Figure 7. Usage of the new long-span composite truss system.

A loaded 4.5m specimen truss with design intumescent fire protection was tested in 2006 at Warrington Fire Laboratories. This behaved very much as predicted by the modelling approach described here, and showed the one-hour fire resistance predicted by the modelling for the intumescent thickness applied. Subsequent development by Metsec Ltd has produced mainly detailed solutions to manufacturing and constructional problems. These have included the development and testing of a unique integrated type of shear connector in the form of plates welded into the upper-chord cap section, and self-sealing ends to the trapezoidal decking. Most significantly, in order to allow a greater flexibility in spans and floor loadings the bottom chord cap section has been replaced by a rectangular hollow section. This gives access to a much greater range of section sizes and wall thicknesses, but in no way changes the behaviour which has been shown in this study.

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