THE BEHAVIOUR OF LIGHTWEIGHT COMPOSITE FLOOR TRUSSES IN FIRE

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ABSTRACT

Despite severe local damage induced by the impact by two hijacked airplanes, the structures of WTC 1&2 were able to sustain 103 and 56 minutes of the subsequent fire on September 11 2001. The purpose of this study is to contribute to the understanding of the behaviour of the twin towers during these events in structural fire engineering terms. A series of numerical analyses were conducted using the FE package *Vulcan*, on the behaviour of the typical long-span composite floor truss in the fire. The composite truss is considered under a variety of scenarios, varying the boundary conditions, the degree of protection and loading. The time-temperature relationship of the steel truss components and the LWC slab have been obtained using Eurocode formulae and thermal analysis software respectively. The results are presented as graphs of deflections against time.

MODELLING

Typical occupancy floors of both WTC 1 & 2 were composed of one-way long-span and short-span composite trusses and two-way corner areas. In the long-span area dual composite trusses, 18.3m in length and 752.8mm in depth, were spaced 2.0m apart. A concrete slab cast onto profiled metal decking was used. Double angles 50.8mm x 38.1mm x 6.4mm (A50) and 76.2mm x 50.8mm x 9.4mm (A36) were used for the top and bottom chords. A solid round bar of 29.0mm diameter (A50) was used for the first web diagonal member. The remaining web diagonals were formed from solid round bar of 27.7mm diameter (A46). The steel decking was placed underneath the lightweight concrete ($20.7 N/mm^2$) slab comprising a 101.3mm topping and 38.1mm ribs. The vertical floor-to-floor spacing was 3.7m. For this study, one half of the composite truss was numerically modelled with and without a supporting column (Fig 1).

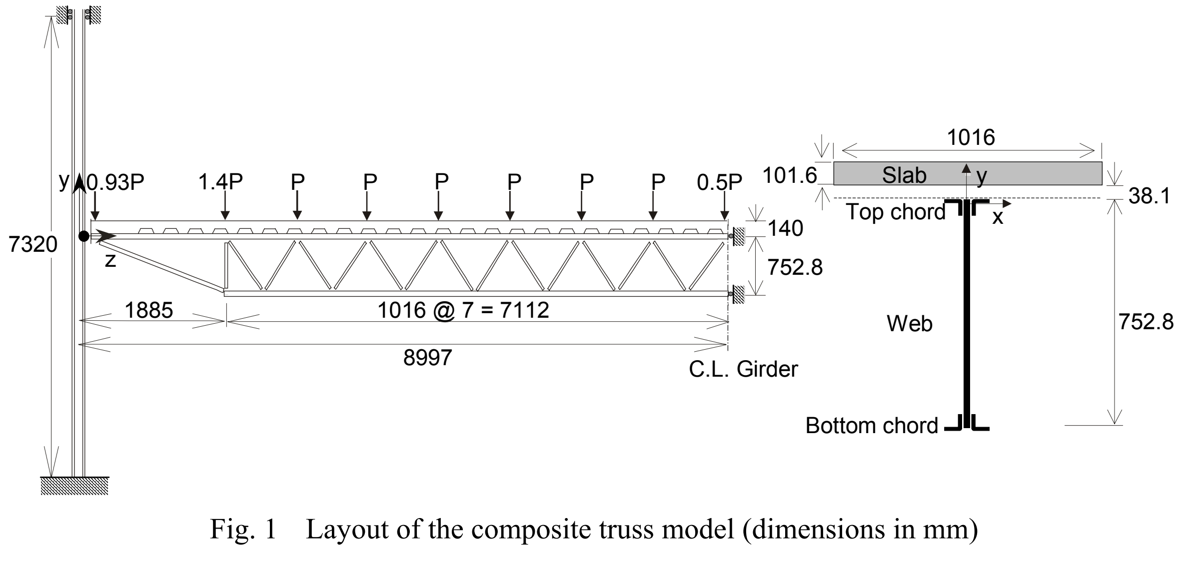

Fig. 1 Layout of the composite truss model (dimensions in mm)

In the case which includes the column, a pinned beam-to-column connection was used. For the isolated truss case a simple support was used as the end boundary condition. The proposed model was designed assuming continuous top and bottom chords and pin-ended web members with no eccentricity at the connections. No local damage was introduced into the model due to the initial impact.

Two types of loading, a design load at fire limit state and an arbitrarily assumed load for the real situation, were considered. The loading at the fire limit state was calculated using office floor loading, factored according to BS5950:Pt8 (BSI 1990) and BS6399: Pt1 (BSI 1996), of 4.8kN/m^2. The arbitrarily assumed load, a combination of the full dead load and one third of the superimposed live load, was 3.9kN/m^2. All transverse loading was applied at the connections of the top chord and bracing members.

TEMPERATURE ASSESSMENT

The FEMA report (2002) estimated that approximately 4000 gallons of airplane fuel may have remained on the impact floors of WTC1. If this is assumed to have been evenly distributed over 5 damaged floors in WTC1, the increase of the fire load due to the fuel may not have been significant different from the generic office fire load (EC1:Pt1.2 1991). The opening factor, dependent on the local aircraft impact damage, varies between floors, but was still relatively low due to the large ratio of floor area to storey height. Therefore, the ISO834 (1975) fire was applied in order to assess the temperature development of the composite truss. The influence of the active sprinkler protection system was not included since this was almost certainly inactive or ineffective. The long-span composite trusses in the twin towers were assumed to be insulated under normal conditions to resist 2 hours of the standard fire by a prescriptive method. Due to the airplane impact and blast, very little of the extremely fragile sprayed insulation material would have remained intact on the surface of many trusses. Hence, an unprotected composite truss was considered as a reasonable lower-bound case.

In the unprotected case, the temperature development of each of the chords (Fig 2) was calculated using the section factor, A_m/V, using the formula provided by EC4: Pt1.2 (ECS 1994). The column was still assumed to be protected.

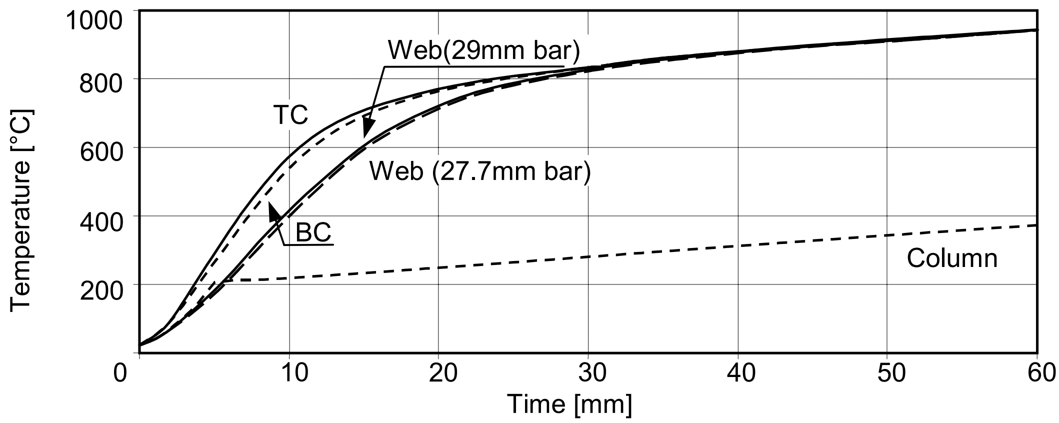

Fig. 2 Temperature development of unprotected chord members in the standard fire

In the case of the protected truss, the temperature development of the protected members was assumed to increase to about 200°C in a similar fashion to the unprotected members, and then to progress linearly up to 620°C at 120 minutes for the top and bottom chords and to 550°C for the web members and the column.

A thermal analysis (Fig 3) was conducted, using thermal analysis software (Huang *et al.*, 1996) to generate the temperature development of the LWC slab, divided into 9 layers with the steel decking, in the standard fire. The moisture content was assumed to be 2.0% of the concrete weight and coefficients of the heat transfer were chosen from generic data given by Purkiss (1996). The material properties of steel and concrete at elevated temperatures were adopted according to EC3:Pt1.2 (ECS 1993) and EC4:Pt1.2 (ECS 1994).

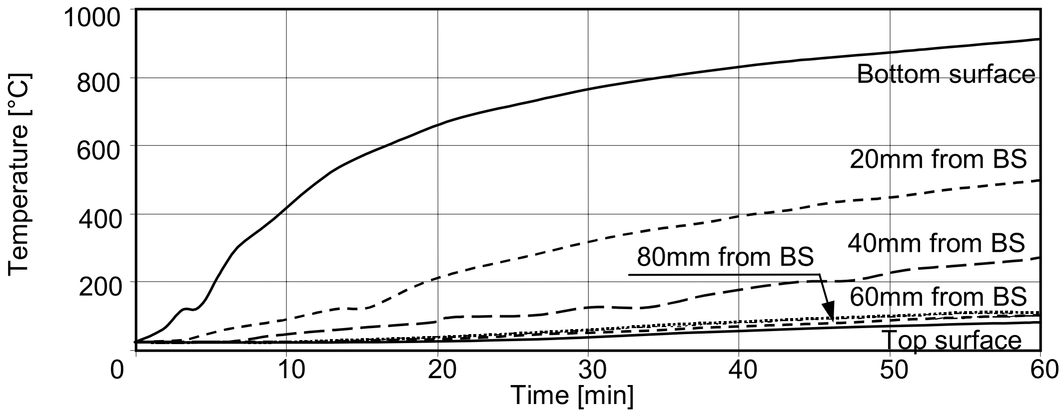

Fig. 3 Temperature development of the 100mm topping of LWC slab in the standard fire

THE UNRESTRAINED COMPOSITE TRUSS

After the aircraft impact on WTC1 it was observed that several floors were simultaneously exposed to a fire caused by the widely-spread jet fuel. In this situation the top and bottom floors in this area might still retain horizontal restraint stiffness, provided by the supporting column, as with a single-storey compartment fire model. However, it is likely that the horizontal restraint to the intermediate storeys would be much lower or non-existent. In order to understand the behaviour of the middle floors in fire, a simply supported composite truss, with and without protection, was numerically analysed, using *Vulcan* for 60 minutes of exposure to the standard fire, under loadings of 4.8kN/m^2 for the fire limit state and 3.9kN/m^2 for the probable actual condition. The equilibrium condition of the composite truss at ambient temperature is illustrated in Fig 4. Double black and white arrows respectively indicate compression and tension components. Single black vertical arrows represent resultant shear forces.

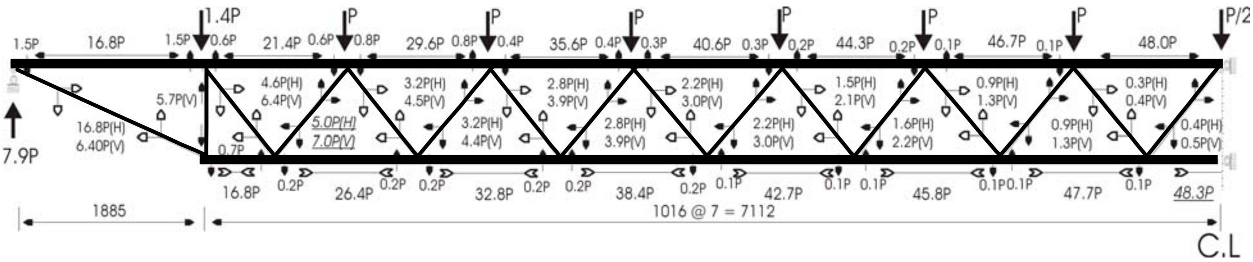

Fig. 4 Equilibrium of the unrestrained composite truss at ambient temperature

The critical elements of the model can be identified at ambient temperature by examining the load ratios of the individual members. The peak value of load ratio under tension occurs in the mid-span member of the bottom chord, 8.5-9.0m from the support. For compression it is the second compressive web diagonal, which is the fourth web member from the support. Numerical analysis results for the protected and unprotected cases are shown in Fig 5.

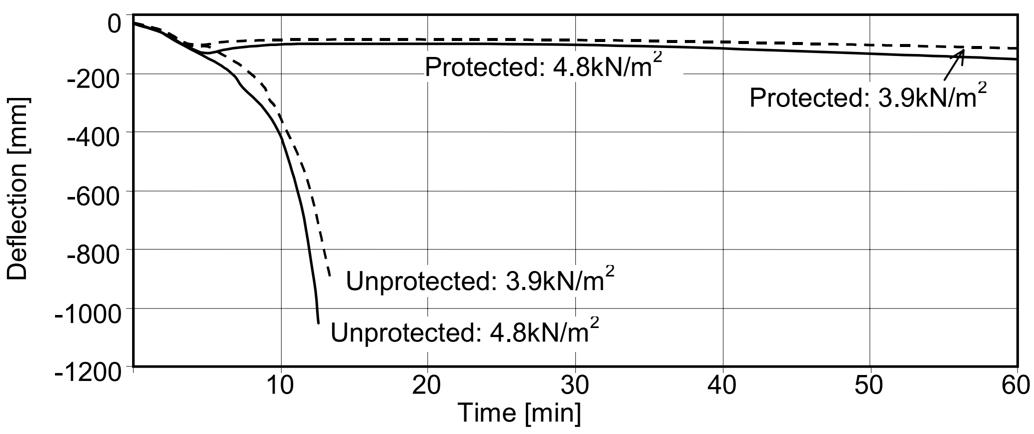

Fig 5. Vertical deflection at top chord mid-span of the simply supported truss in the standard fire

The protected composite truss, under the loadings 4.8kN/m^2 and 3.9kN/m^2 respectively, deflects approximately L/100 and L/150 at 60 minutes of the standard fire without any local instability. The unprotected trusses resist up to 12.5 and 13.4 minutes, at which time the second compressive web member buckles. The equilibrium force pattern in the simply supported composite truss in the standard fire was the same as that at ambient temperature. The structural resistance was not sensitive to the level of the loads in the standard fire for these models.

THE COMPOSITE TRUSS WITH A SUPPORTING COLUMN

To simulate the top and bottom floors of the fire-exposed levels, the structural behaviour of protected and unprotected composite trusses with a supporting column was investigated under the 4.8kN/m^2 and 3.9kN/m^2 loadings for 60 minutes' exposure to the standard fire. The performance of the models is shown, in Fig 6, in terms of the vertical deflection of the mid-span top chord against time.

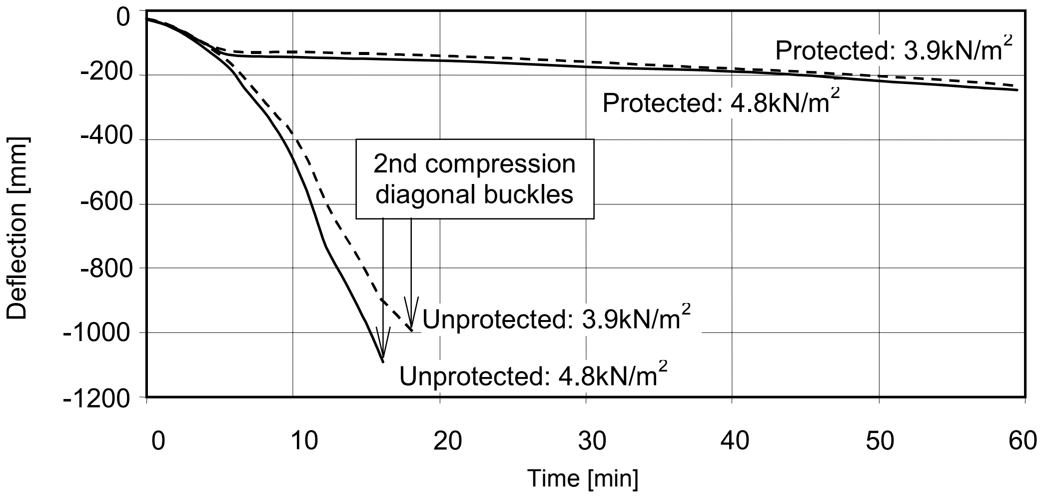

Fig. 6 Vertical deflection at the mid-span of the top chord in the standard fire.

The variation of the horizontal reaction at the beam-column connection, under the arbitrary loading, is plotted in Fig 7. Fig 6 shows that the protected models resist 60 minutes of the standard fire with deflection of about L/90. During this period, the column consistently experiences a push-out force (Fig. 7) due to thermal elongation of the slab and top chord.

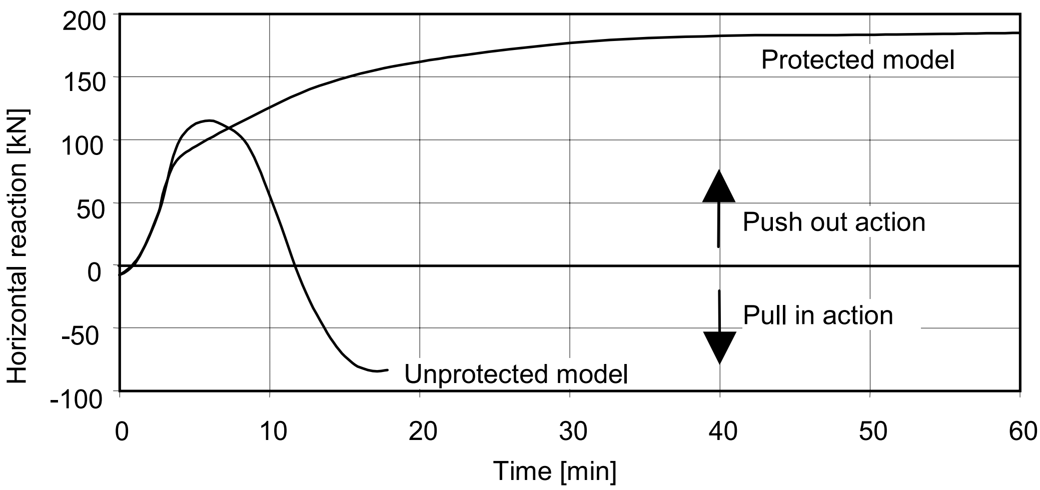

Fig. 7. Horizontal reaction at the beam-column connection under estimated loading in the standard fire.

The force pattern within the protected composite truss in fire is initially very similar, as illustrated in Fig. 8(a) to that at ambient temperature.

The unprotected models under 4.8kN/m^2 and 3.9kN/m^2 loadings are shown in Fig 6 to have their first local instability at 16.1 and 18.0 minutes of the standard fire. The second compressive diagonal, which was seen to have the highest compressive load-ratio in the simply supported condition, is the first to buckle. Through a progressive load redistribution process, illustrated in Fig. 8(b), the compressive diagonals then successively buckle at the same temperature. Eventually, this series of local instabilities causes the remaining part of the composite truss to collapse through tension of the slab and top chord. It can be seen from Fig 7 that the horizontal reaction changes in direction from outward to inward at 11.7 minutes of the standard fire. This occurs as yielding spreads from mid-span outwards in the members of the bottom chord. Once this condition occurs, the moment resistance of the composite truss, generated by the lever arm between top and bottom chords, begins to cease to carry the majority of the load. The load-carrying mechanism changes progressively to catenary action, shown in Fig. 8(c), in which both the slab and top chord carry most of the imposed load in tension rather than in balanced compression and tension with the bottom chord. The additional fire resistance which would normally be generated by the catenary action may not be fully utilised in this unprotected model, due to the effect of the simultaneous local instabilities causing most of the catenary tension to be concentrated in the top chord and slab.

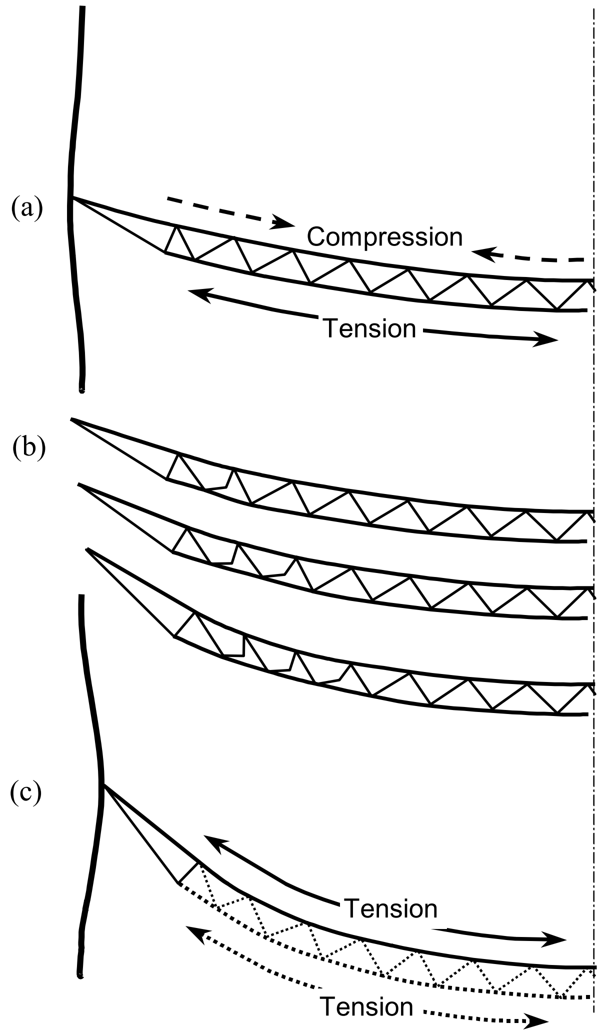

Fig 8. Equilibrium of the composite truss: (a) in bending; (b) as diagonals buckle; (c) in full catenary action.

The analyses showed that the performance of the protected and unprotected composite trusses was not very sensitive to the load level, in terms of either deflection or resistance period. Clearly a rapid reversal of the reaction force was induced at the beam-column connection during the heating process for the unprotected model. It is clearly possible that this connection, which represents those at the highest and lowest floors affected by fire, may have been unable to sustain the tension force when its own tying strength was depleted by the elevated temperature.

CONCLUSION

During the events of September 11 2001 the twin towers, after undergoing massive structural damage, both survived in fire for over 55 minutes. This study was aimed at understanding the performance in fire of the floor support system employed, in terms of the effects of passive protection, support conditions and loading.

With respect to the passive fire protection, 2-hour protected and unprotected cases were considered. In the unprotected situation, temperatures in the individual truss elements were calculated using the incremental method described in EC4: Part 1.2 and assuming a standard (ISO834) fire curve. The concurrent temperature profiles through the concrete slab were obtained from thermal analysis. For the protected case, the temperature rise of the truss steelwork was assumed to increase to 200°C as with the unprotected case, and then to develop linearly to 550°C for web members and 620°C for the chords at 120 minutes.

Numerical analyses were conducted using *Vulcan*, of the composite trusses with and without accounting for the restraint provided by a supporting column, simply representing the extreme and the middle floors among the fire exposed levels, up to 60 minutes of the standard fire. The numerical analyses demonstrated that the protected composite trusses would resist 60 minutes of the standard fire within a deflection of L/100, under loadings of $4.8kN/m^2$ and $3.9kN/m^2$. Unprotected simply supported trusses initially lost stability at 12.5 and 13.4 minutes due to buckling of the second compressive web diagonal. Both of the protected trusses with a supporting column deflected approximately L/90 at 60 minutes of the standard fire without any local instability occurring. The unprotected composite truss with a supporting column was shown to resist for 16.1 and 18.0 minutes of the standard fire before the progressive buckling of web compression diagonals caused a loss of stability. This would undoubtedly have re-stabilised when catenary action of the top chord and slab reinforcement took effect, but their tensile strength, together with the tying strength of the beam-column connections, would then become critical. For both types of support condition the behaviour of the unprotected truss was relatively insensitive to the level of loading.

Functions, such as the severity of the fire, loading conditions, the influence of the remaining protection pattern and the connection robustness, still need to be further investigated if composite trusses similar to those used in WTC1 and 2 are to be routinely used in long-span flooring applications.

Acknowledgment:

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