The Influence of Tensile Membrane Action in Concrete Slabs on the Behaviour of Composite Steel-framed Buildings in Fire

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Abstract

A computer program VULCAN has been progressively developed for some years at the University of Sheffield, with the objective of enabling three-dimensional modelling of the behaviour of composite buildings in fire. In this paper the theoretical basis of the non-linear layered procedure used to model the reinforced concrete floor slabs, which includes both geometric and material non-linearity, is briefly outlined. Several of the full-scale fire tests carried out in 1995-96 on the composite frame at Cardington, representing cases in which different degrees of in-plane restraint are provided by the adjacent structure, are modelled to evaluate the influence of tensile membrane action in the concrete slabs on the structural behaviour in fire. In order to illustrate the influence of membrane action and its relationship with boundary restraint, all cases have been analysed using both geometrically linear and non-linear slab elements. A series of parametric studies has been carried out as an initial investigation into the characteristics of steel reinforcement which allow this action to take place. It is clear that the influence of tensile membrane action of concrete slabs on the behaviour of such composite structures in fire is very important, and should be accounted for in later additions and amendments to structural fire engineering design codes.

Introduction

In 1995-96 six large fire tests were carried out on a full-scale composite multi-storey building at the BRE Fire Research Laboratory at Cardington UK. The test results confirmed that steel members in real multi-storey buildings have significantly greater fire resistance than isolated members in the ISO834 (1985) Standard Fire Test. Although steel temperatures became considerably greater than Eurocode4 (1992) critical temperatures for the loading levels used, no run-away failures were observed. A general observation was that the composite slab appears to play a significant part in preventing structural collapse, and in consequence it is very important to model the action of concrete floor slabs correctly in fire.

The numerical software VULCAN (Najjar and Burgess, 1996; Bailey *et al.*, 1996; Huang *et al.*, 1999, 2000a, 2000b) has been developed in recent years at the University of Sheffield for three-dimensional analysis of the structural behaviour of composite and steel-framed buildings in fire. The objective of the current phase of the research has been to extend the layered procedures previously developed by the authors (Huang *et al.*, 1999, 2000b) for the modelling in fire of solid reinforced concrete slabs and orthotropic slabs, including their ribbed lower part, to include geometric non-linearity. A total Lagrangian approach is adopted throughout, in which displacements are referred to the original configuration.

In this paper the theoretical basis of the non-linear layered procedure used to model the reinforced concrete floor slabs, which includes both geometric and material non-linearity, is very briefly outlined. Several of the Cardington full-scale fire tests are modelled, to evaluate the influence of tensile membrane action in the concrete slabs on the structural behaviour in fire. A series of parametric studies has been carried out as the first stage of an investigation of the slab design parameters which control this action.

Theoretical Basis of the Non-linear Layered Procedure

In this research concrete slabs are modelled as an assemblage of finite plate elements. Each element is modelled using the quadrilateral 9-noded higher-order isoparametric elements developed by Bathe (1996). The plate elements are sub-divided into several layers representing concrete and distributed reinforcing steel. The main assumptions for the layered approach can be summarised as follows:

- The slab elements are considered to be made up of plain concrete layers and reinforcing steel layers, and there is no slip between layers.
- Each layer can have a different temperature, but within each layer the temperature is uniform. The initial material properties of each layer may be different, and the stress-strain relationships may change independently for each layer.
- The reinforcing steel bars in one of the orthogonal mesh directions are modelled by an equivalent steel layer with stiffness only in the direction of the reinforcement. The thickness of each steel layer is such that its cross-sectional area is equal to the total area of the corresponding reinforcing bars. There can be many such reinforcing layers in the longitudinal and the transverse directions. Perfect bond is assumed between the reinforcing steel layers and the surrounding concrete.
- Concrete layers are in a state of plane stress, and concrete is considered to be orthotropic after cracking.

A smeared cracking model has been used, in which cracking at any Gauss point is identified layer by layer. After the initiation of cracking in a single direction, the concrete is treated as an orthotropic material with principal axes normal and parallel to the cracking direction. Upon further loading of singly-cracked concrete, if the tensile stress in the direction parallel to the first set of smeared cracks exceeds the maximum tensile strength then a second set of cracks forms. After crushing concrete is assumed to lose all stiffness. The uniaxial properties of concrete and reinforcing steel at elevated temperatures, which are specified in Eurocode4 (1992) have been adopted in this model. This development is intended to enable VULCAN to model the membrane action caused by *"P-Δ"* interaction of concrete floor slabs in fire. The details of the formulations of this non-linear layered procedure can be found in Huang *et al.* (2000c).

Modelling of the Restrained Beam Test

A full-scale eight-storey composite test building was constructed by BRE at its Cardington Laboratory during 1994 to resemble a modern city-centre medium-rise office development typical of current UK practice. Composite action was achieved between both primary and secondary steel beams and the floor slabs using shear studs. The Restrained Beam Test was carried out by British Steel plc (Bentley, 1995a, 1995b) on the frame in January 1995. It involved heating a single secondary beam and an area of the surrounding slab on the seventh floor. The member tested consisted of a 305x165UB40 section spanning between columns D2 and E2, and was heated using a specially constructed gas-fired furnace along the middle 8m of its 9m length. The location of the test is shown in Figure 1. The extent of the structure incorporated within the model is also indicated in Figure 1.

In the Cardington test building the total nominal thickness of the composite slabs was 130mm, with a 75mm continuous portion at the top, giving effective stiffness factors of 0.72 and 0.34 parallel and perpendicular to the rib direction. The ambient-temperature material properties used in the modelling, based on test values where these were available, were as follows:

- The yield strength of steel was 308MPa for Grade 43 steel (S275) and 390MPa for Grade 50 steel (S355);
- The yield strength of the steel used in the anti-crack mesh was assumed to be 460 MPa;
- The elastic modulus of steel was 2.1×10^5 MPa;
- The average compressive strength of concrete test samples was 35 MPa, and this is used.

A uniform floor load of 5.48kN/m^2 was applied to the whole building, using sand-bags for the imposed portion, and this is assumed in the modelling. The temperature distributions across the section of the steel beam and the depth of the slab are assumed to be invariant with position within the fire compartment. These temperature distributions at any time in the test are the averages of the recorded test temperature distributions at this time across the beam and slab. The maximum recorded temperatures of the bottom flange, web and top flange were 834°C, 816°C, and 764°C respectively, and the maximum recorded temperatures of the bottom and top layers of the slabs were 481°C and 129°C respectively. In order to investigate the structural behaviour of the restrained beam up to extremely high temperatures, these temperatures have been extrapolated linearly, so that the maximum temperatures of the bottom flange of the beam and the bottom layer of the slab become 1005°C and 637°C respectively. The temperature of the bottom flange of the tested beam is used as the "key temperature" which is quoted in all figures.

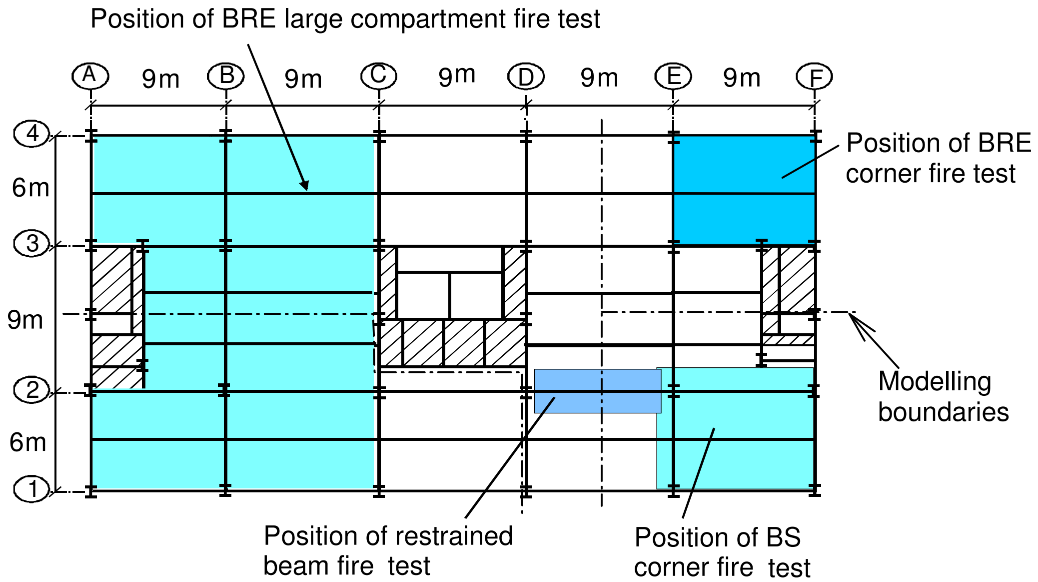

Figure 1. Locations of the four fire tests in the Cardington test frame.

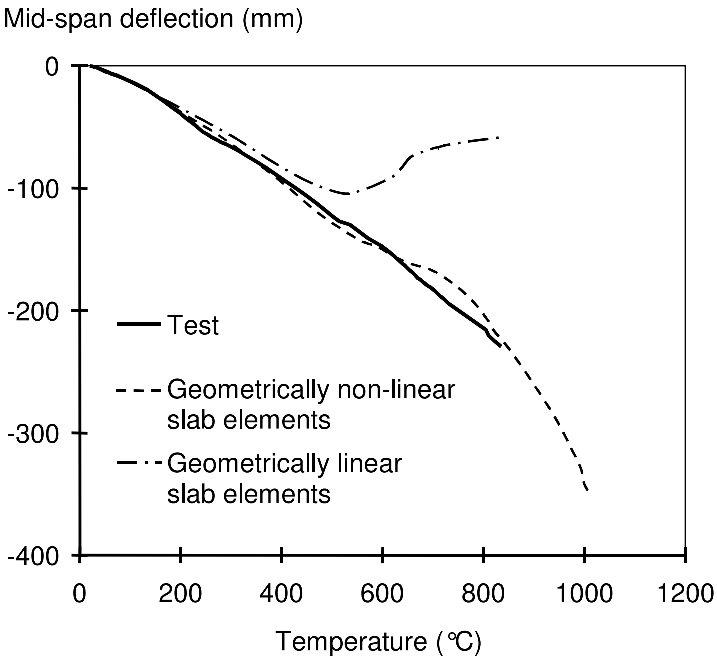

Figure 2. Comparison of predicted and measured deflections for the Restrained Beam test using geometrically linear and non-linear slab elements.

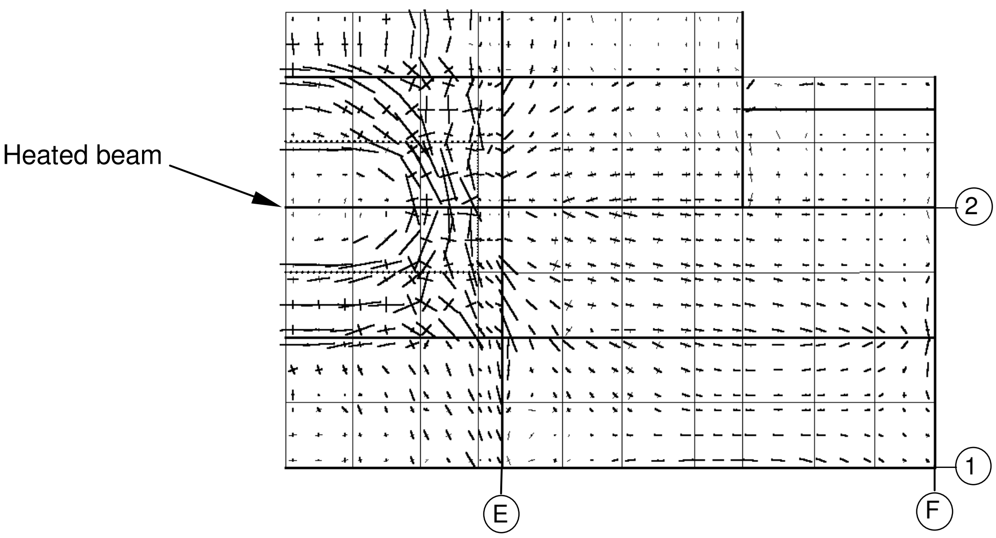

Figure 3. Distribution of the principal membrane tractions at 1000°C for the Restrained Beam test (thick line = compression; thin line = tension).

The test is modelled using geometrically linear and non-linear slab elements, and the orthotropic nature of the slabs is included in the modelling. The test results showing the variation of mid-span deflection of the heated beam against the bottom flange key temperature are given in Figure 2, together with the analytical results. It is evident that the influence of membrane action is very significant in this situation of high restraint, especially when the key temperature is higher than 500°C. The predictions of the present model, in which geometric non-linearity of the slab element is included, are in remarkably good agreement with the test results. Due to considerable vertical support from cool structure surrounding the heated zone the second-order forces caused by the slab within the fire compartment acting as a doubly-curved shell become very significant and eventually dominate the structural behaviour.

Figure 3 shows the principal membrane tractions within the slabs at very high temperature. It can be seen that very high compressive tractions are formed around the edges of the fire compartment, and within the fire compartment the compressive tractions at the edges gradually change to tensile in the central areas. The central zone of the fire compartment is subject to tensile membrane forces which are carried mainly by the anti-crack mesh. If the tensile forces are large enough to cause the steel reinforcement to fracture then the slabs within the fire compartment will fail, at least in terms of maintaining the integrity of the compartment. However, because of the massive in-plane restraint in this case the dominant membrane tractions are peripheral compressions.

Modelling of the British Steel Corner Fire Test

In July 1995 Test 3 of the British Steel series was carried out (Bann *et al.*, 1995; Bentley *et al.*, 1996) on a corner bay of the structure 9.98m wide by 7.57m deep. The walls of the fire compartment, shown schematically in Figure 1, were constructed using lightweight concrete blockwork, the top of which was detached to allow free deflection of the structure above. All columns and perimeter beams were wrapped with ceramic fibre, but all other structural elements were left unprotected. The test was fired using timber cribs giving an overall fire load of 45kg/m^2 to produce a natural fire. During the fire test the maximum recorded atmosphere temperature in the compartment was 1028°C, which occurred after 80 minutes. Steel temperatures and structural deflections were recorded at key locations and at required intervals throughout the test, providing information for comparison with analytical results.

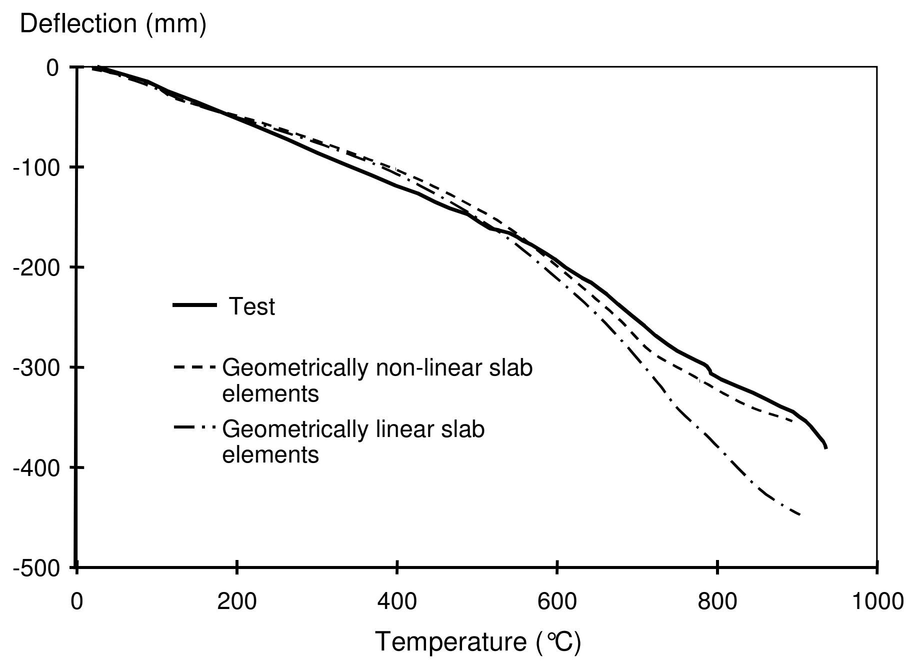

Figure 4. Comparison of predicted and measured deflections for the BS Corner Fire Test using geometrically linear and non-linear slab elements.

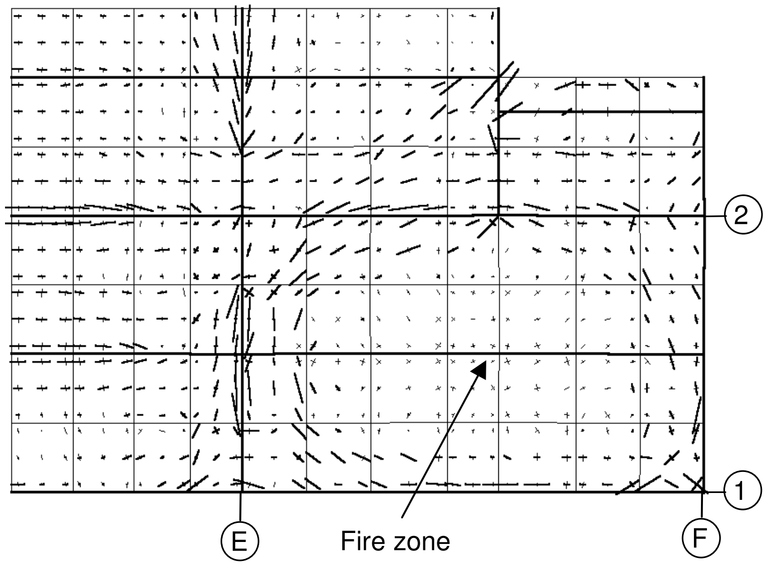

Figure 5. Distribution of the principal membrane tractions at 900°C for the BS Corner Fire Test (thick line = compression; thin line = tension).

The material properties at ambient temperature and the uniform floor load were the same as in the Restrained Beam test. From the test temperature data the temperature profiles of the beams and columns and temperature distribution through the thickness of the concrete slab within the fire compartment are rationalised for the modelling. Again the test has been modelled using both geometrically linear and non-linear slab elements. Figure 4 shows the test and the predicted deflections for the mid-span of the central secondary beam of the fire compartment, against the bottom flange temperatures of the beam. The distribution of the principal membrane tractions at 900°C is shown in Figure 5.

It can be seen from Figure 4 that when the vertical deflections were less than 300mm, in which range the temperatures of the steel beam were less than 700°C, there was little influence of geometric non-linearity. After further increase of temperature the steel beams had lost most of their strength, and the loads above fire compartment were largely carried by the floor slab. In this corner test little in-plane restraint was provided by surrounding cool structure, and so the in-plane tractions within the corner bay need to be almost self-equilibrating. This means that the tensile membrane forces within the central zone of the floor slab were balanced by the ring of compression forces formed around the perimeter of the fire compartment. The deflection of the slabs was decreased significantly in the high-temperature range due to this tensile membrane action, in which the anti-cracking mesh is the key component. This phenomenon is confirmed visually in Figure 5.

The area of steel reinforcing mesh used in the concrete slabs of the Cardington tests was 142mm^2/m. In order to demonstrate the effect of reinforcement on the structural behaviour two fictitious reinforcing meshes, with half and double the actual area, were employed for comparison, and the geometrically non-linear slab element was used. The predictions and test results for the vertical deflection at mid-span of the central secondary beam are shown in Figure 6. It can be inferred that when the temperature of steel beam was above 500°C the tensile membrane forces within the floor slabs were mainly carried by the steel mesh. Hence the load capacity of concrete slabs depends strongly on the reinforcement area and strength. In an extreme situation, when slabs are subjected very large deflections, the reinforcement will become a key element in maintaining the integrity of the structure, and if the reinforcement fails this may initiate a major structural integrity failure.

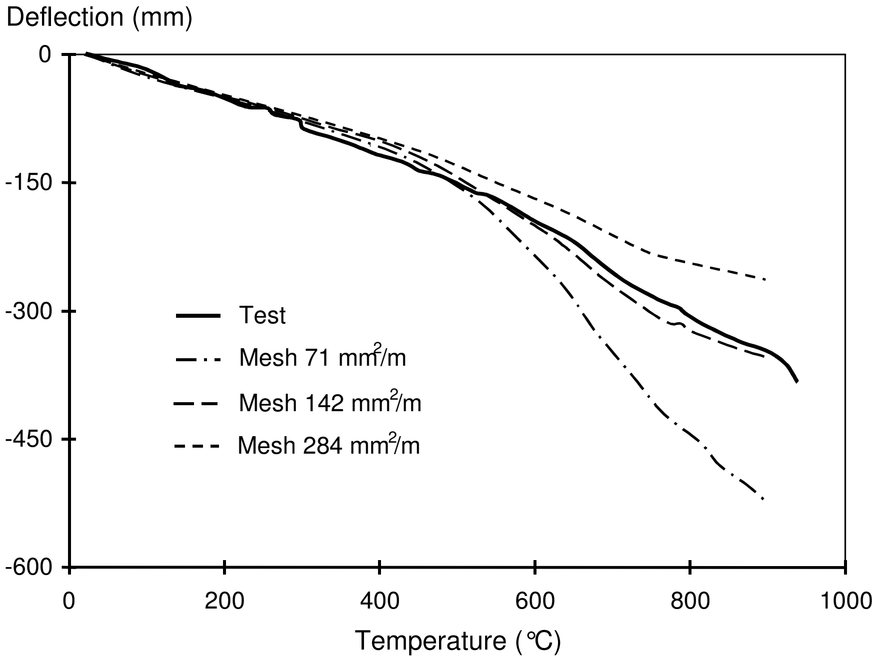

Figure 6. Comparison of predicted deflections with different areas of reinforcement against test results for the BS Corner Fire Test.

Modelling of the BRE Corner Fire Test

In October 1995 BRE carried out their first fire test (Lennon, 1995) on the Cardington Test Frame, in another corner bay between the second and third floors using a natural fire generated by timber cribs. Its location within the floor-plan is shown in Figure 1. The internal boundaries of the fire compartment were constructed using fire-resistant board which was placed centrally on grid-lines E and 3. A full-height concrete-block wall formed the building perimeter wall on grid-line F, with a one-metre high concrete-block dado wall topped by double-glazing units forming the boundary on grid-line 4. The glazing system was supported by two equally spaced wind-posts (150x75x10RSA) attached to the edge beam. The more important wind-posts were those in the storey above the fire compartment, fixed between the heated edge beam, to which they were bolted using slotted holes allowing up to 80mm of vertical movement, and the underside of the unheated edge beam of the storey above. Thus, vertical downward movement of the heated beam was practically restricted to 80mm. The compartment was totally enclosed at the start of the fire, with all windows and doors closed.

A fire load of 40kg/m^2 of timber was provided by twelve cribs placed on the second floor, over the floor area of 54m^2, giving a total fire load of 2160kg in the compartment. All of the steel beams were left unprotected. The steel columns inside the compartment were protected by insulating material up to the underside of the ceiling slab, including the beam-to-column connections. In the initial stages of the fire test there was very slow fire growth due to very restricted ventilation, and flash-over was delayed until after one window had been broken from the outside. During the fire test the maximum recorded atmosphere temperature in the centre of the compartment was 1051°C at 102 minutes. The maximum average measured steel temperatures, of the bottom flanges of the central secondary beam and the edge beam were 842°C and 590°C respectively at 114 minutes. The temperature distributions through the thickness of the slab were recorded at various sampling points around the slab and the maximum average measured temperature of the bottom layer of the concrete slab was about 285°C at 114 minutes.

The extent of the structure incorporated within the numerical model is shown in Figure 1. The ambient-temperature material properties and uniform floor load were the same as previous tests mentioned. In order to investigate the structural behaviour of this corner fire test up to extremely high temperatures the measured temperatures have been extrapolated linearly in the modelling:

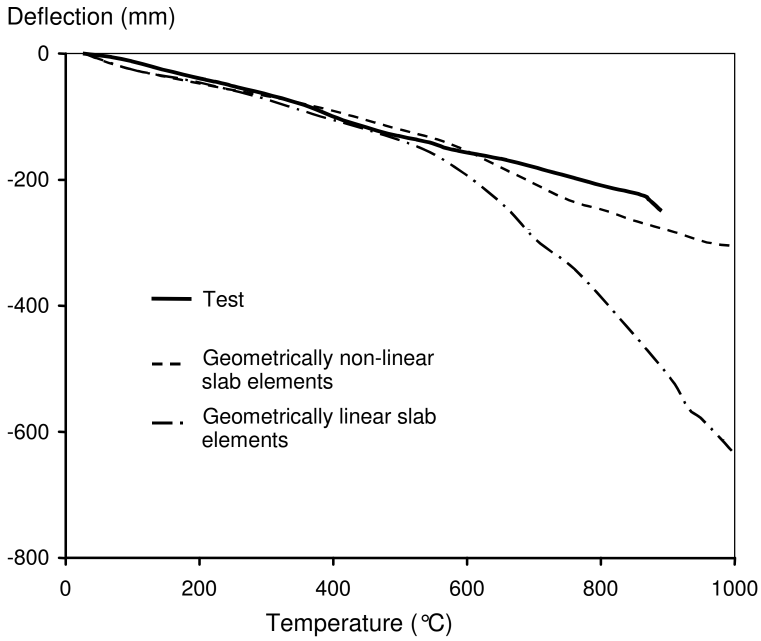

Figure 7. Comparison of predicted and measured deflections for the BRE Corner Fire Test using geometrically linear and non-linear slab elements.

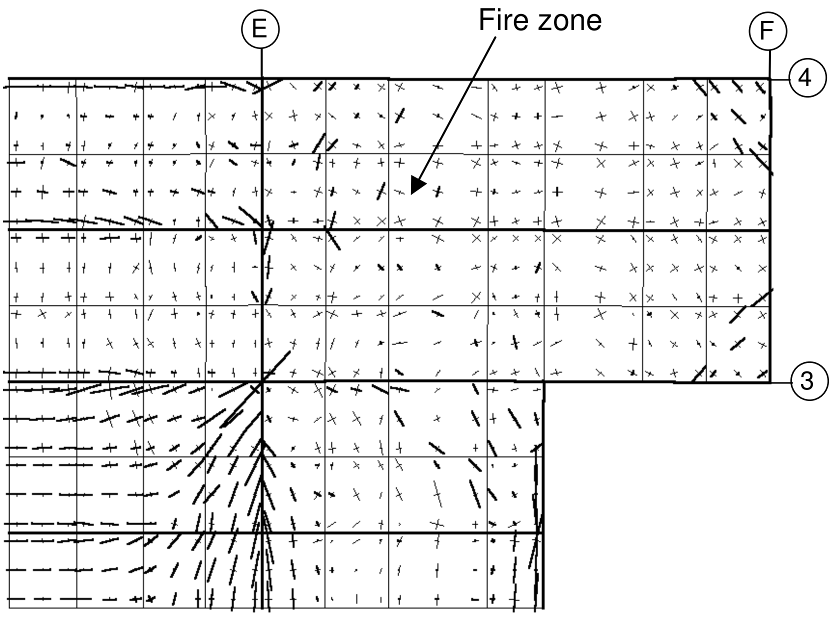

Figure 8. Distribution of two principal membrane tractions at 1000°C for BRE Corner Fire Test (thick line = compression; thin line = tension).

In order to demonstrate the influence of membrane action the test was modelled using both geometrically linear and non-linear slab elements. The comparisons between the predicted and test results for vertical deflections of the mid-span of the central secondary beam within the fire compartment are shown in Figure 7. Figure 8 shows the distribution of the principal membrane tractions in the slab at 1000°C.

It can be seen that when the vertical deflections were less than 200mm, in which range the temperatures of the steel beam were less than 600°C, geometric non-linearity had little influence. After further increase of temperature the steel beams had gradually lost most of their strength, and the loads above fire compartment were largely carried by the floor slab. As in the British Steel Corner Fire Test little in-plane restraint was provided by surrounding cool structure. It is evident that in this case geometric non-linearity was more significant than in the BS Corner Fire Test. The fire compartment was slightly smaller than in the BS test and the boundary walls were constructed just beneath the perimeter beams of the fire compartment. Also the two wind-posts provided support for the edge beam on the front of the building. The four perimeter beams of the compartment therefore had only a limited capacity for vertical deflection. This means that the concrete slab within the fire compartment was close to having all four edges vertically supported. Hence the slab was forced to deform in double curvature, thus generating a more significant degree of tensile membrane action.

From Figure 8 it can be seen that the typical compression ring was not actually formed along the edges of the slabs of the compartment, although these can be seen in part at the slab corners. This is mainly due to the relatively larger temperature differences (about 300°C) between edge beams and the slab in this case, as well as the relatively low temperature of the edge beams when the central secondary beam is very hot. The steel edge beams attempt to expand against the cooler concrete slabs and, since they retain a reasonable proportion of ambient-temperature strength, thus generate high compression forces. In consequence the tensile membrane forces generated within the central zone of the slabs can be almost wholly equilibrated by these peripheral compressions in the steel beams without assistance from the adjacent concrete. The high-temperature deflection of the slabs seems again to have been decreased significantly due to this tensile membrane action, in which the anti-cracking mesh is the key component.

Modelling of BRE Large Compartment Fire Test

In April 1996 BRE carried out their second fire test (Lennon, 1996) on the Cardington Test Frame, in a large compartment between the second and third floors. This extended across the full width of the building, between grid-line A and a line 0.5m from grid-line C (see Figure 1). This is thought to be the largest fully instrumented fire test which has taken place to date, covering an area of 340m^2. A fire load of 40kg/m^2 of wood was provided by timber cribs. All the internal steel beams were left unprotected, but the columns were protected over their full height, including their connections. The maximum recorded atmosphere and steelwork temperatures were 763°C and 691°C, respectively. The average maximum temperature of the slab soffit was about 260°C. The extent of the structure incorporated within the numerical model is shown in Figure 1, and more detail, together with the finite element mesh layout adopted, is shown in Figure 9.

The ambient-temperature material properties assumed were the same as for the other tests, and a uniform floor load of 5.48kN/m^2 was again assumed. In order to investigate the structural behaviour of this large fire test up to extremely high temperatures the measured temperatures have been extrapolated linearly, so that the following assumptions have been made for the modelling:

- Beams B2, B3, B4, B6, B7, B8, B10, B11 had identical temperature distributions, with the maximum temperatures of the bottom flange, web and top flange being 1000°C, 957°C and 885°C respectively;
- Beams B1, B5, B9 and B14 had the same temperature distributions, with the maximum temperatures of the bottom flange, web and top flange being 640°C, 640°C and 567°C respectively. They were also supported vertically by walls and by the wind-posts.
- The cross-sections of all columns inside and surrounding the fire compartment had uniform temperature at any time, with a maximum temperature of 92°C;

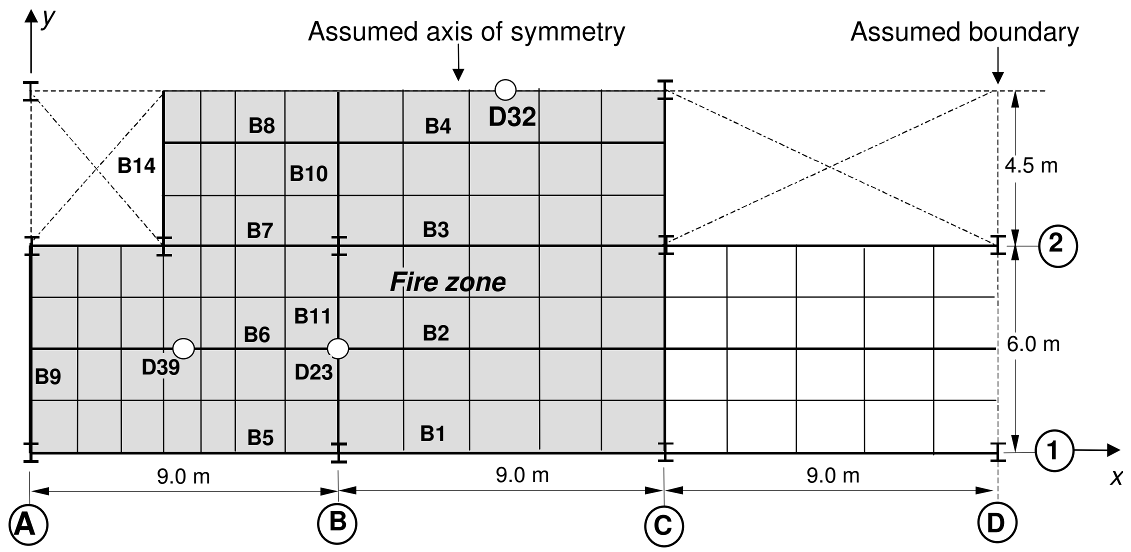

Figure 9. Finite element layout adopted in the analysis of the BRE Large Compartment Fire Test, together with the locations of comparisons.

- The average temperature distribution through the thickness of the concrete slab was used, with the maximum temperatures of bottom and top layers being taken as 366°C and 106°C respectively.

In order to demonstrate the influence of membrane action the test was modelled using both geometrically linear and non-linear slab elements. The comparisons between the predicted and test results for vertical deflections taken at the 2 key positions D32 and D39 shown in Figure 9 are plotted in Figures 10 and 11. It can be seen from Figure 11 that the predictions are in good agreement with test results at position D39, but Figure 10 shows some discrepancies concerning position D32, for which the predictions are in good agreement with test results up to 600°C, beyond which the program produces less deflection than the actual test data. In this test, because of the large amount of instrumentation the sand-bags used to add imposed load had to be moved at some locations, so that a fully uniform floor load could not really be guaranteed. With such a deep fire compartment, ventilated at both sides, it is also impossible to be certain that the heating was uniform at any time or across the whole area. In spite of these uncertainties the program produces very reasonable modelling compared with the test results for this large-scale fire test.

Figure 12 shows the predicted distribution of principal membrane tractions at a steel key temperature of 1000°C, and the deflection profile of the composite slab is shown in Figure 13 at the same temperature, including the smeared cracking patterns of the top concrete layer.

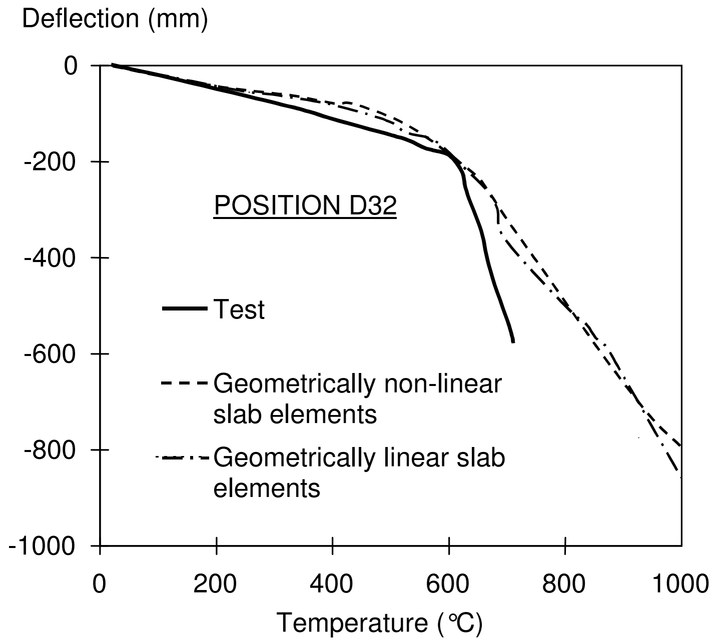

Figure 10. Comparison of predicted and measured deflections at position D32 for BRE Large Compartment Fire Test using geometrically linear and non-linear slab elements.

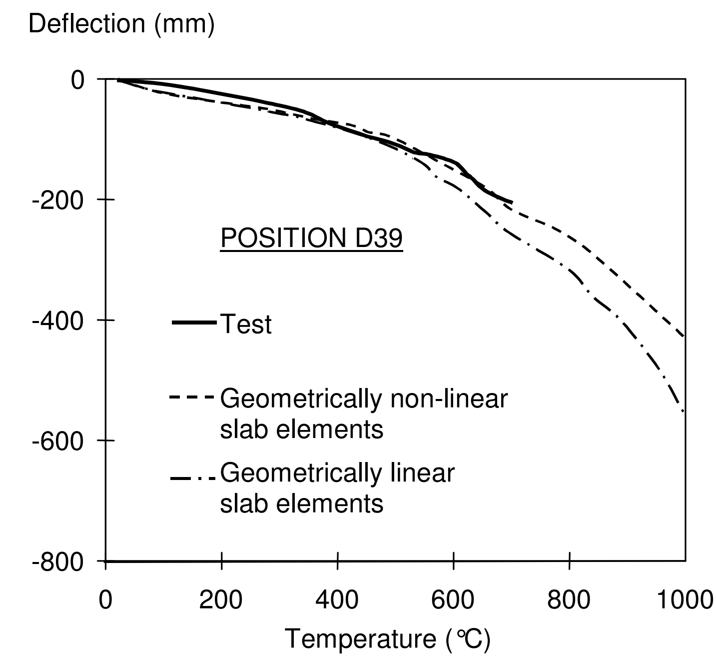

Figure 11. Comparison of predicted and measured deflections at position D39 for BRE Large Compartment Fire Test using geometrically linear and non-linear slab elements.

From Figure 10 it can be seen that the influence of geometric non-linearity of the slab elements is small. The reason for this is that the floor slab essentially deforms in single curvature in the region of position D32 (see Figure 13). The restraint provided by some structural bracing in the stair-well and the atrium at the ends of the compartment was ignored in the modelling. Hence relatively small tensile membrane forces are generated within the slabs above the fire compartment. Comparing the results for this position with those presented in Figure 11 for position D39 reveals some effect of geometric non-linearity, especially when the key temperatures are above 800°C and the deflections of primary beam B11 are increasing sharply. At this stage the floor slab at position D39 is forced to some extent to deform into double curvature, and thus the influence of membrane action becomes more significant.

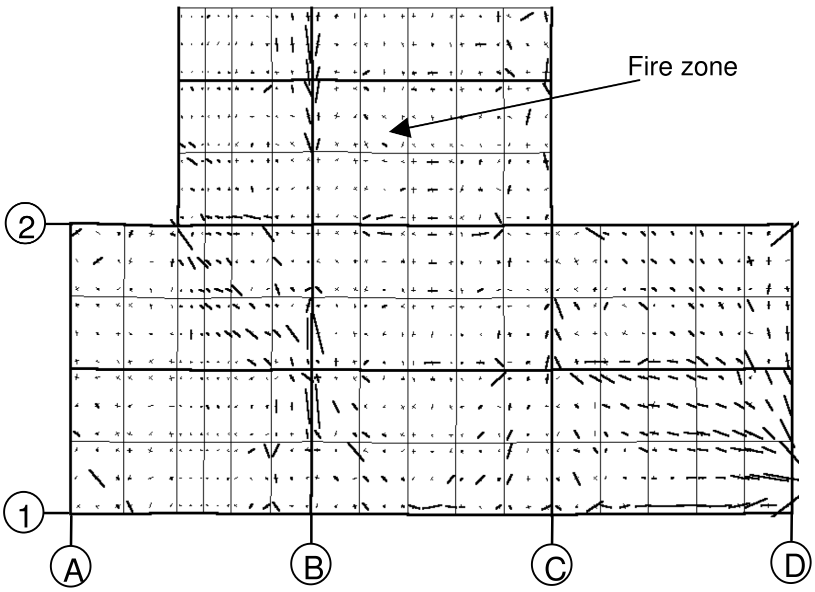

Figure 12. Distribution of the principal membrane tractions at 1000°C for BRE Large Compartment Fire Test (thick line = compression; thin line = tension).

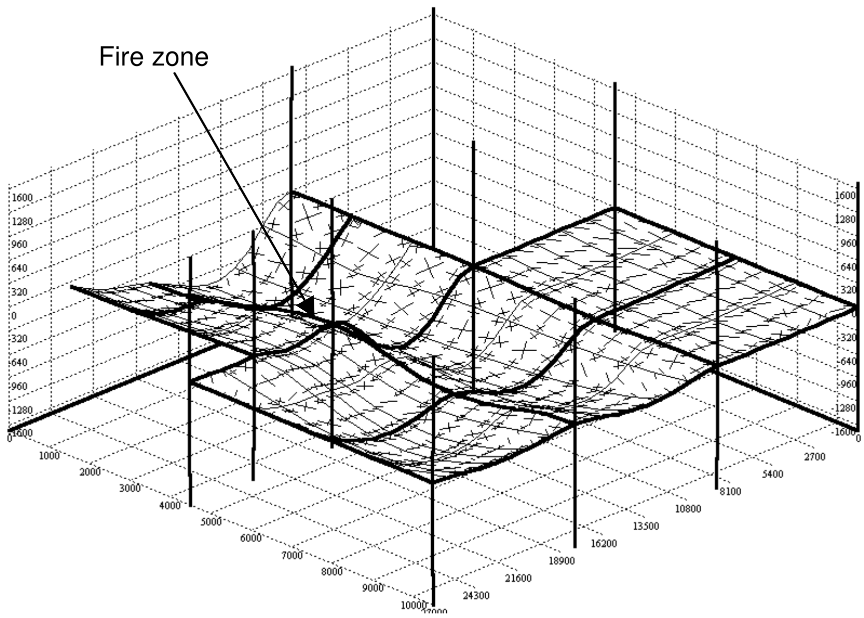

Figure 13. Deflection profiles at 1000°C for BRE Large Compartment Fire Test, with cracking patterns of top layer of floor slab.

Conclusions

The basic features of the non-linear layered procedure for modelling of membrane actions of reinforced concrete slabs and their influence on the structural behaviour of composite steel-framed buildings in fire have been described. Four full-scale fire tests carried out by British Steel plc (now Corus Group plc) and BRE on the Cardington composite test frame have been modelled. Some preliminary conclusions can be drawn as follows:

- When the temperatures of exposed steel beams are less than 300-400°C the influence of geometrically non-linear behaviour of concrete slabs on the structural behaviour of a composite building is small, but when temperatures are above 500°C it increases greatly. It then becomes very important to model the behaviour of concrete slabs correctly.
- When the deflections of floor slabs become large the influence of tensile membrane action can become important in supporting the slab loading. Whether this is capable of preventing final fracture of the slabs depends mainly on the area and strength of the steel reinforcement used and the degree to which it is insulated from temperature rise by its concrete cover, even if this is cracked.
- It has been seen that tensile membrane action is not necessarily dependent on in-plane restraint, since the mid-slab tensions can be balanced by peripheral compressions, provided that the slab is forced to deform in double curvature because of vertical support at its edges. The generation of tensile membrane action in the floor slabs is minimised when they deform in single curvature, or when folding mechanisms are allowed due to inadequate vertical support to some edges. If this is the case, then slabs may undergo much higher deflections before the tractions reach critical values at which fracture occurs, and this will cause considerable pulling-together of the edges. In these circumstances the relative movement of the column-ends may become unacceptable, or unsafe, and tensile fracture of slabs will not be the critical failure mechanism. Restraint to pull-in from adjacent structure could then become an important factor in utilising the strength of the reinforcement and reducing deflections.

The locations of these four full-scale fire tests cover a wide range of restraint to horizontal movement of slabs or beams. It is apparent that the slab reinforcement, which has been employed mainly for control of cracking during the curing process and may not be subject to any structural calculation in design, may be vitally important in ensuring integrity when tensile membrane action is mobilised in the fire limit state. Design of this reinforcement for tensile membrane action in fire may be the price to be paid for design processes based on the real performance in fire of such systems rather than that of isolated elements under artificial test conditions.

Acknowledgement: The authors gratefully acknowledge the support of the Engineering and Physical Sciences Research Council of Great Britain under grant GR/M99194.

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