THE ULTIMATE BEHAVIOUR OF COMPOSITE FRAMES IN FIRE CONDITIONS [1]

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Abstract

Large-scale fire tests in the UK appear to confirm that, in real buildings, unprotected composite beams have a significantly greater fire resistance than when furnace-tested as isolated members. This is due to interactions between the heated structure within the fire compartment and adjacent cool structure. The computer program *Vulcan* has been developed at the University of Sheffield to model the behaviour of composite and steel framed buildings in fire. In this paper two large-scale fire tests, with very different degrees of restraint provided by the adjacent structure, are modelled to show how this restraint affects the behaviour within the heated zone. It is evident that the influence of membrane action is important if the integrity of fire compartments is to be maintained. Where high boundary restraint is present the second-order forces caused by geometric non-linearity in the slab within the fire compartment become very significant and eventually dominate the structural behaviour. A large generic composite framed building has also been designed, and a series of analyses have been carried out on fire compartments of different extents and locations, to assist in understanding the interactions between cool and hot zones of the composite structure.

Key words: composite structures; structural behaviour in fire; membrane actions.

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1. INTRODUCTION

In 1995 and 1996 six large-scale fire tests conducted at Cardington in the United Kingdom appeared to confirm a long-held suspicion that composite beams with unprotected downstand steel sections have a significantly greater fire resistance when incorporated in the floors of real buildings than when they are tested as isolated members. Although steel temperatures became considerably greater in these tests than the Eurocode 4 (1992) critical temperatures for the load levels imposed, no run-away structural failures were observed. It was noticed that composite concrete slabs appeared to play an important part in preventing structural collapse. It was therefore very important that the phenomena involved should be understood, and that analytical methods should be developed to model the behaviour of such structures when subjected to fire.

A computer program *Vulcan* (Najjar & Burgess 1996, Bailey 1995, Huang *et al.* 1999) 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. In this paper two large-scale fire tests, using compartments subjected to very different degrees of restraint to thermal expansion by the adjacent structure, are modelled to show how this restraint affects the structural behaviour within the heated zone. To examine the influence of tensile membrane action in slabs at the high deflections generated, and its relationship with boundary restraint, these tests have been analysed using both geometrically linear and non-linear slab elements. A large generic composite steel-framed building has also been designed, and a series of analyses have been carried out on fire compartments of different extents and locations. The results are compared with fire resistance calculations according to EC4.

2. MODELLING OF THE RESTRAINED BEAM TEST

The 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. The Restrained Beam Test was carried out by British Steel plc (Bentley *et al.* 1995) on the frame in January 1995. It was the first, and the smallest, of the six fire tests carried out in 1995 and 1996, and involved heating a single secondary beam and an area of the surrounding concrete slab on the seventh floor. The member tested consisted of a 305x165UB40 steel 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 Fig. 1. The extent of the structure incorporated within the model is also indicated in Fig. 1, with a more detailed representation including the finite element mesh layout in Fig.2. In the Cardington building the total nominal thickness of the composite slabs was 130mm, with a 70mm continuous portion above the ribs, giving effective stiffness factors of 0.72 and 0.34 for bending parallel and perpendicular to the rib direction respectively. The ambient-temperature material properties used in the modelling were based on test values.

The whole building was subjected to a uniform dead+imposed floor loading of 5.48kN/m^2 using sandbags as the imposed component of load, 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 slab were 481°C and 129°C respectively.

In order to investigate the structural behaviour of the restrained beam up to extremely high temperatures, the test temperatures have been extrapolated linearly, so that the maximum temperatures of the bottom flange of the beam and bottom layer of the slab become 1005°C and 637°C respectively. In the following text the temperature of the bottom flange of the tested beam is used as the key temperature which is quoted in all figures.

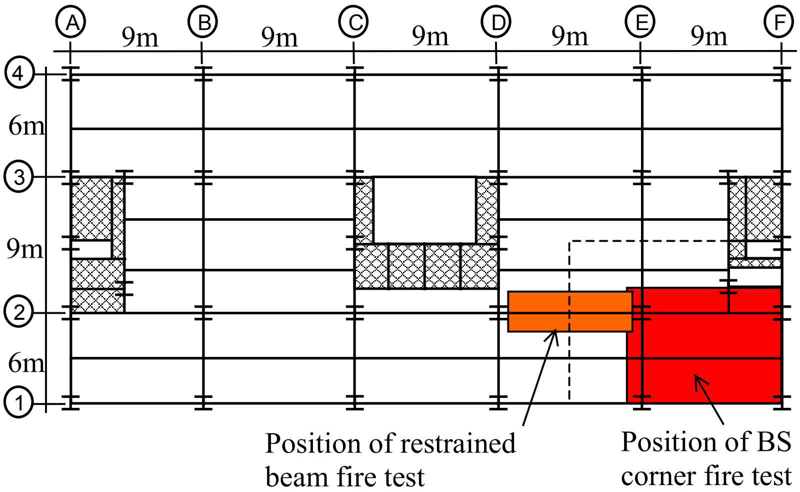

Fig. 1 Locations of the two Cardington fire tests studied in this paper.

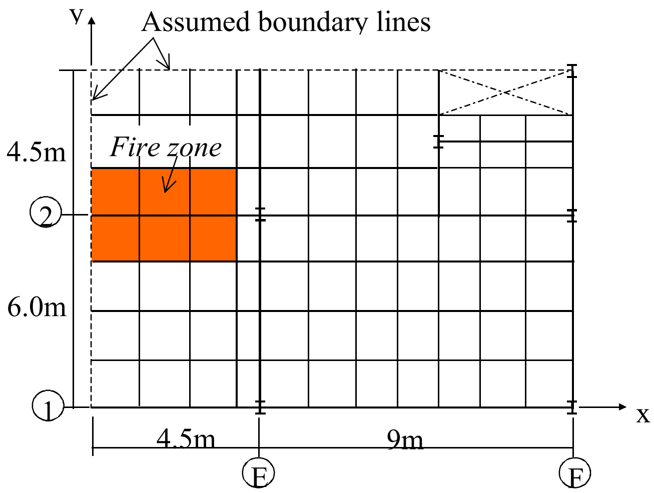

Fig. 2 Finite element layout for analysis of the Restrained Beam test.

The test has been modelled using both geometrically linear and non-linear slab elements, and the orthotropic nature of the slabs is included in the modelling. The test results, plotted in terms of the mid-span deflection of the heated beam against the bottom flange key temperature are shown on Fig. 3, together with the analytical results. It is evident that the influence of geometrically non-linear effects is very significant in this situation of high restraint, especially when the key temperature is higher than 500°C. The predictions of the model in which geometric non-linearity of slab element is included are in remarkably good agreement with the test results. Due to the high restraint to horizontal movement at the edges of the heated zone the second-order forces caused by geometric non-linearity in the beam and slab within the fire compartment become very significant and eventually dominate the structural behaviour.

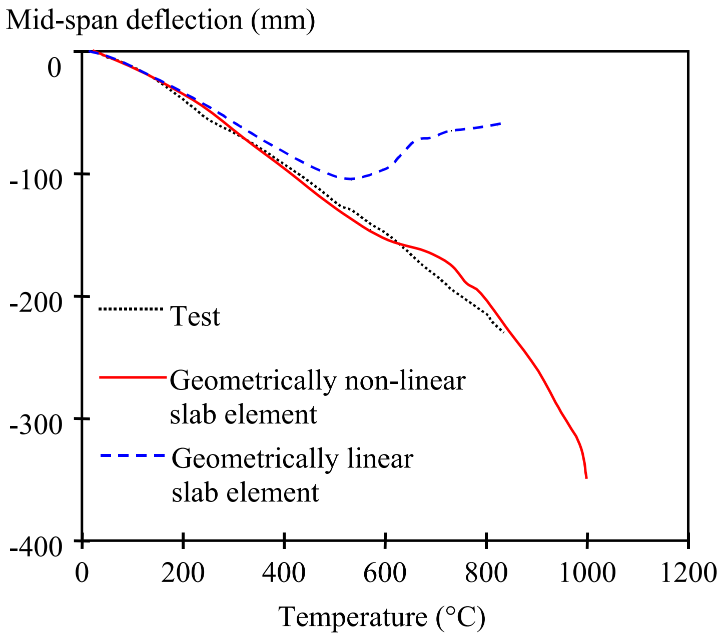

Fig. 3 Comparison of predicted and measured deflections for the Restrained Beam Test using geometrically linear and non-linear slab elements.

Fig. 4 shows the distributions of the two principal membrane tractions in the slab at ambient temperature. It can be seen that the slabs above the secondary and primary beams act very much in line with the normal engineer's assumption for the flanges of composite beams, being essentially in compression parallel to the beam. This reduces somewhat in the areas mid-way between parallel beams due to the well-known phenomenon of shear lag. At the beam-ends, which are zones of hogging action, these slab membrane tractions are in tension. In contrast, the membrane tractions within the slabs at very high temperature are presented in Fig. 5. It can be seen that very high compressive tractions are formed surrounding 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.

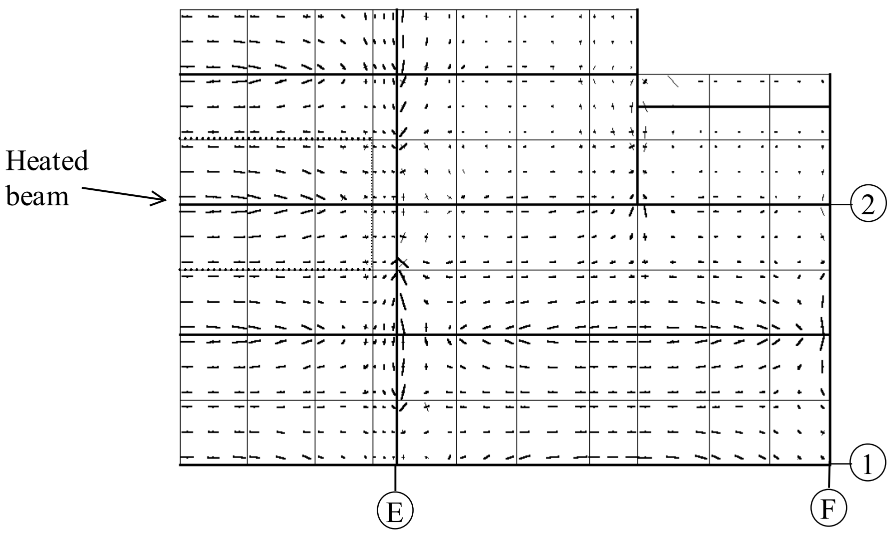

Fig. 4 Distribution of two principal membrane tractions at 20°C for the Restrained Beam test (thick line = compression; thin line = tension).

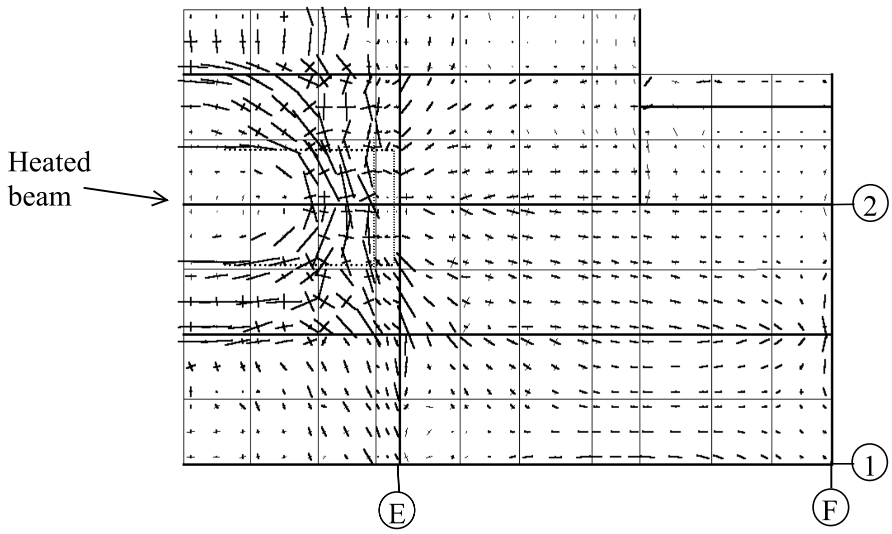

Fig. 5 Distribution of the principal membrane tractions at 1000°C for the Restrained Beam test (thick line=compression; thin line=tension).

3. MODELLING OF THE BRITISH STEEL CORNER FIRE TEST

In July 1995 the third fire test of the British Steel series was carried out (Bentley et al. 1996), on a corner bay of the Cardington test frame 9.98m wide by 7.57m deep. All columns and perimeter beams were wrapped with ceramic fibre, but all other structural elements were left unprotected. During the fire test the maximum recorded atmosphere temperature in the compartment was 1028°C, which occurred after 80 minutes. The test location and the extent of the structure incorporated within the numerical model are shown in Fig. 1, and the finite element mesh layout is shown in Fig. 6.

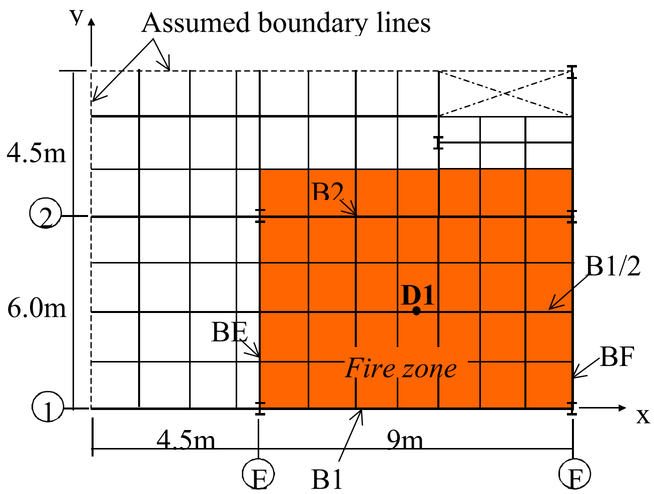

Fig. 6 Finite element layout for analysis of the Corner Fire Test, together with the locations of comparisons.

The floor load was the same in all tests. In order to rationalise the test temperature profiles of the beams and columns the following assumptions were made (see Fig. 6 for the beam positions): Unprotected beams B1/2, B2, BE have the same temperature distributions, in

which the maximum temperatures of the bottom flange, web and top flange are 900°C, 860°C, and 800°C respectively. Protected beams B1 and BF have the same temperature distributions in which the temperatures of the bottom flange, web and top flange were 250°C, 180°C, and 110°C respectively. The cross-sections of all protected columns were assumed to have identical uniform temperature distribution with a maximum temperature of 160°C. The mean test temperature distributions through the thickness of the concrete slab were used, with the maximum temperatures of bottom and top layers at 360°C and 70°C respectively.
Again the test was modelled with both geometrically linear and non-linear slab elements. Fig. 7 shows the test results, plotted with the predictions for deflection at the mid-span of secondary beam B1/2 (point D11) against the bottom flange temperatures of the beam.

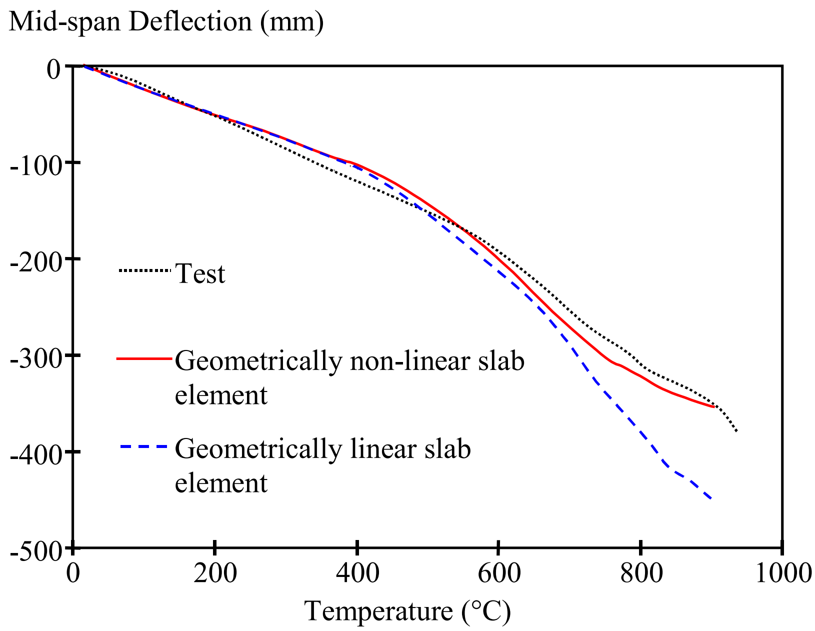

Fig. 7 Comparison of predicted and measured deflections for the BS Corner Fire Test using geometrically linear and non-linear slab elements.

The distribution of the two principal membrane tractions at 900°C is shown in Fig. 8. It can be seen that when the vertical deflections were less than 300mm, for which the temperatures of the steel beam were below 700°C, the influence of slab non-linearity was low. After further temperature increase the heated steel beams retained only a very small percentage of their ambient-temperature strength, so that the loads on the floor slab above the fire compartment were mainly carried by the slab itself. The bending strength of the slab alone was insufficient to carry the applied load, so membrane tension in the middle region must have been the dominant load-resisting mechanism towards the end of the test. In this test it is clear that very little restraint was provided at the two exposed edges of the structure, so the membrane tension within the floor slabs above the corner bay must have been self-equilibrated. The load carrying capacity of the slabs was increased significantly by utilising its tensile membrane capacity, for which the anti-crack mesh is a key component. This is further confirmed in Fig. 8; as the temperature of the steel beam increases its strength and stiffness decrease, so the influence of mesh reinforcement becomes more and more significant. Eventually, when slabs are subjected very large deflections, the mesh becomes the key element to maintaining the integrity of the slab, and when it fractures this will cause a failure of compartmentation. To ensure that fracture of slabs does not occur may necessitate either higher reinforcement ratios or placement of the mesh further from the heated surface of the concrete.

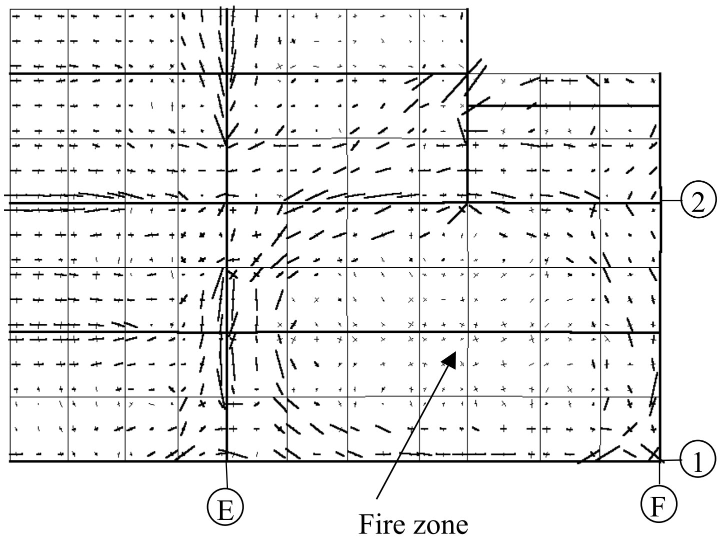

Fig. 8 Distribution of the principal membrane tractions at 900°C for the Corner Fire Test (thick line=compression; thin line=tension).

4. MODELLING OF A LARGE GENERIC COMPOSITE STEEL FRAMED BUILDING

In order to consider the effects shown above without the local detailing of the Cardington building, a large generic composite steel framed building, which has a footprint of 18m by 54m was designed (see Fig. 9). This is assumed to be braced against sway in its central zone.

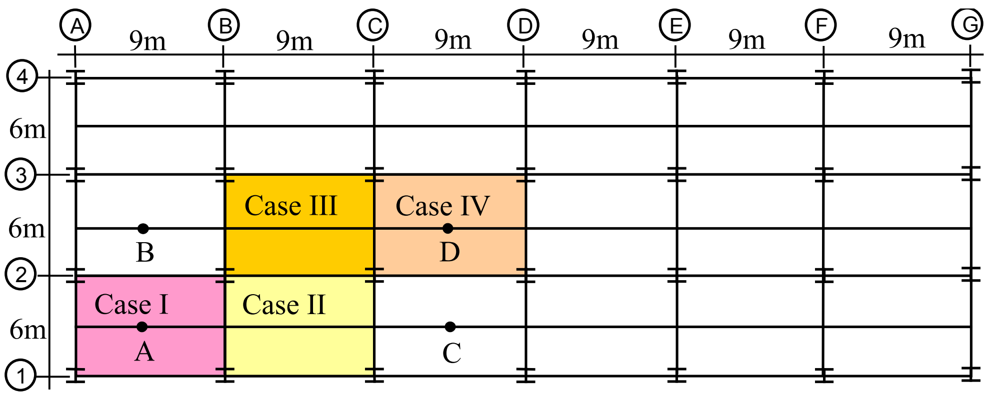

Fig. 9. A large generic composite non-sway steel framed building with different fire compartment locations marked.

Along the length there are 6 bays, each of 9m width, and across the depth there are 3 bays of 6m width. The construction of the building is similar to that of the Cardington test frame but with a higher uniform floor loading of 8.86kN/m^2 in the fire limit state. All primary and secondary beams have been standardised, using 305x165x40UB and 356x171x51UB sections respectively. The load level of all internal secondary beams is 0.7 according to EC4 (1992). The floor slab is identical to that used in the Cardington frame, comprising a profiled metal decking acting compositely with 130mm total depth (including profile) of reinforced concrete slab, incorporating an A142 anti-cracking mesh. A series of analyses have been carried out based on fire compartments of different extents and locations. Temperature distributions corresponding to the development of a typical natural fire (using the temperature records from the Cardington "British Steel Corner Fire Test") have been applied. The results have been compared with normal fire resistance calculations according to EC4 (1992).

In these analyses compartment fires in four different locations have been modelled (see Fig. 9). The strategy is to select single-bay compartments with different degrees of edge restraint, given by continuous or free edges, and finally to consider how the behaviour in these compares with that in an un-compartmented fire which covers the entire floor area. It is assumed that beams on the column grid, and the columns themselves, have been protected but that the other secondary beams are left unprotected. The maximum temperatures of the bottom flange, web and top flange of the unprotected secondary beam within the fire compartment are respectively 1000°C, 960°C, and 890°C. All edge beams of the compartment have the same maximum temperature distributions, with the bottom flange, web and top flange at 280°C, 200°C, and 120°C respectively at this point. All columns have uniform temperature distribution, with a maximum temperature of 160°C. The average test temperature distribution through the thickness of the concrete slab is used, with the maximum temperatures of bottom and top layers at 400°C and 75°C respectively. In the following text the key temperature quoted in all figures refers to the temperature of the bottom flange of the hottest beams.

Fig. 10 shows the mid-span deflection against temperature of the secondary beam within the fire compartment for the four compartments, of which Cases I and IV are the extremes. The influence of surrounding cool structures on the behaviour of the fire compartment is clearly significant, especially when temperatures are high. The restraint from the cool structure and continuous floor slabs provide some benefit in increasing the fire resistance of beam in the fire compartment. For a given deflection level, say *span*/30, the key temperature in Case IV is about 150°C higher than that of Case I.

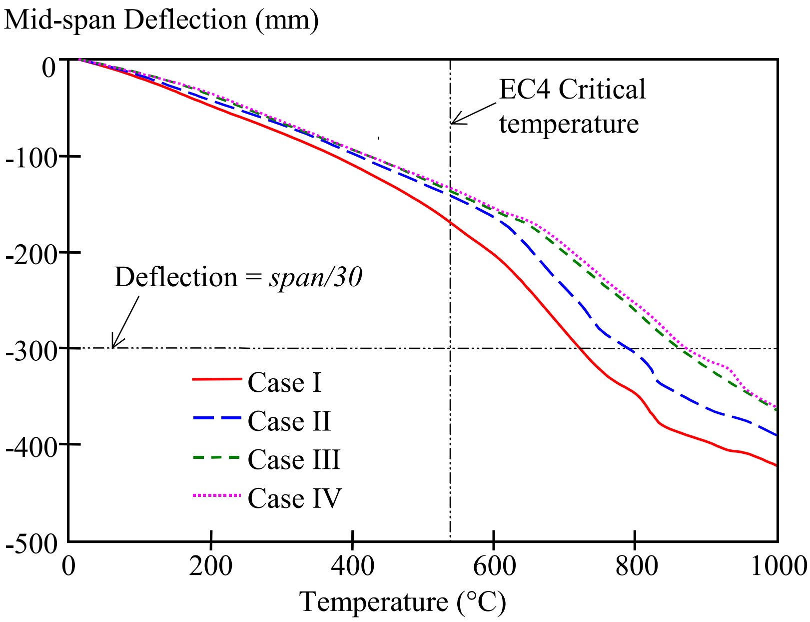

Fig. 10 Comparison of predicted deflections; different fire compartment positions.

To demonstrate the influence of the surrounding edge beams Cases I and IV were re-analysed with all edge beams unprotected, so that the temperatures of all heated beams are identical. The results are shown in Fig. 11. It can be seen that the effect of edge beams is significant. It is interesting to see that when temperatures are less than 500°C there is little difference between the two situations, but when temperatures are over 600°C a huge difference can be seen.

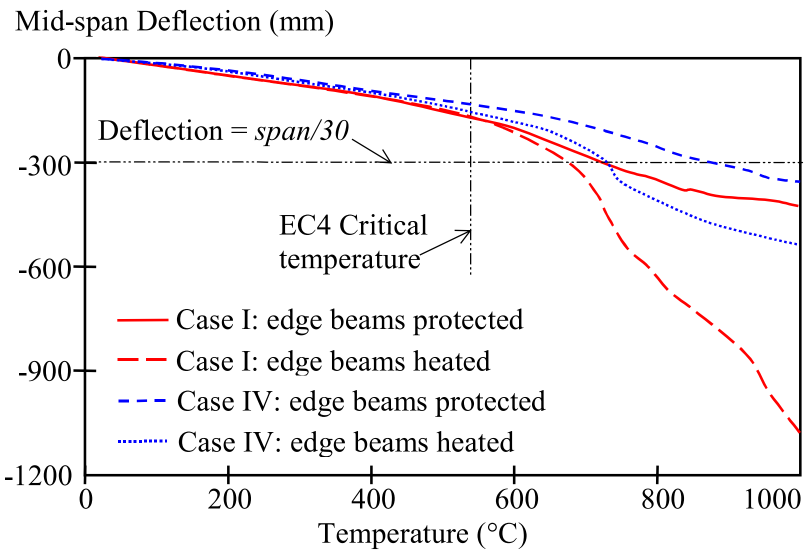

Fig.11 Comparison of predicted deflections; edge beams protected or heated.

Finally, the whole floor of one storey has been modelled in fire without any compartmentation and with all members unprotected. Because of the inherent symmetry of the frame only a quarter of the area has been considered. Fig. 12 shows the deflections of some key points (see Fig. 9) within the structure analysed. When temperatures are higher than 700°C a run-away failure clearly happens, which implies an overall structural failure.

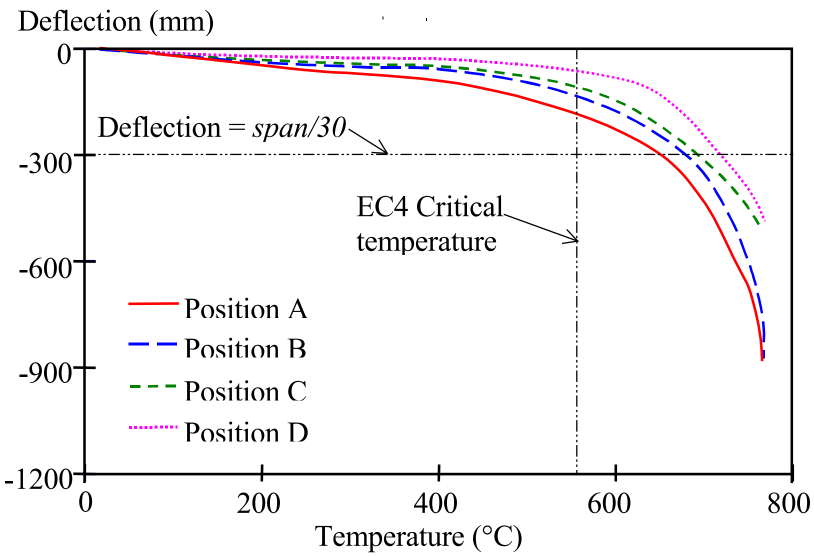

Fig.12 Whole storey heated: deflections at four key positions marked on Fig. 9.

EC4 Part 1.2 deals with fire resistance of composite structures and members. The heated secondary beam within the fire compartment is now idealised in the usual design fashion, as a simply supported composite beam with an effective-width concrete flange. In EC4 terms the load level is $\eta^*=0.7$ and the adaptation factor $\kappa=0.855$, giving a critical temperature of 555°C. This critical temperature is marked on the figures, together with the *span/30* deflection level which is the lower limit used in standard furnace testing. It can be seen that in all situations the temperatures of the bottom flange of the beam at deflection of *span/30* are higher than the EC4 critical temperature. The temperatures at predicted deflections of *span/30* for Position A are always within 17% of the EC4 critical temperature, and always conservative.

5. CONCLUSIONS

It is clear, both from the Cardington tests themselves and from the Vulcan analyses presented here, that unprotected composite beams survive to much higher temperatures in fires when they form part of a building than when tested in isolation. This enhancement to their fire resistance is mainly apparent for steel temperatures above 500°C, and comes from two main influences of the concrete floor slab with which they are continuous:

- As the steel loses strength the slab can form a bridge in bending perpendicular to the line of the beam. This is particularly important for systems at relatively low load levels.
- As the deflections of floor slabs become large the influence of tensile membrane action in the slabs can become very important in supporting the floor loading. Whether this is capable of preventing final fracture of the slabs depends mainly on the amount and properties 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.

Both of these effects depend on the presence of vertical support to the edges of the slab of which the composite beam is a part, reasonably close to the unprotected beam. This explains the very different results given by the generic frame cases in which the edge beams of edge fire compartments are protected and unprotected. It is notable that the case of an overall fire, in which all beams are unprotected and the restraint level is low, gives a runaway failure which is similar to that of a simply supported beam, but at a significantly higher temperature.

It is now clearly appropriate to raise serious questions about the validity of structural fire design codes which base fire resistance calculations on standard furnace tests of isolated members. Future developments must allow a more realistic modelling of real behaviour to be used by designers who wish to optimise fire resistance strategies.

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