ANALYSIS OF COMPOSITE FLOORS WITH DIFFERENT FIRE PROTECTION REGIMES SUBJECT TO COMPARTMENT FIRES

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ABSTRACT

A large generic composite flooring system with a footprint of 36m x 36m has been designed. A series of analyses has been performed, based on different patterns of fire protection to the downstand steel beams. It is evident that the presence or absence of tensile membrane action in the concrete slabs is a major influence on the ultimate integrity of the flooring system at high distortions, and that the extent to which it occurs depends largely on the pattern of fire protection adopted for the steel downstand beams. The analyses include different extents and locations of fire compartments, in order to develop a better understanding of the interactions between the cool and hot zones of the composite flooring system. It is clear that the surrounding structure has significant influence on the behaviour of composite floors within fire compartments, and that tensile membrane action can be utilised in fire resistant design provided that certain key members are protected. The behaviour is compared against the critical temperatures for 30 and 60 minutes' fire resistance, calculated according to BS5950 Part 8, according to which all beams require fire protection for both fire resistance periods. It is shown that a significant proportion of the steel beams can be left unprotected for either fire resistance period. The analyses also demonstrate the enhancements to fire resistance which can be achieved by increasing the size of the reinforcing mesh, increasing the size of the steel beams, or using different fire protection strategies.

KEYWORDS

Tensile membrane action, composite structures, fire protection, compartment fire, fire resistance.

INTRODUCTION

In the current British design standard BS5950 Part 8 (British Standards Institution 1990), and also in the draft Eurocode EC3 Part 1.2 (European Committee for Standardisation 1993) designers are encouraged to treat fire as one of the basic design limit states for steel framed structures. Calculation methods given in these documents take account of:

- Non-uniform heating, due to partial protection which is either inherent in the framing system used or is retrospectively added to the members,
- The level of loading at the fire limit state (by partial safety factors appropriate to the probabilities associated with these accidental conditions),
- Realistic stress-strain characteristics of steel at elevated temperatures.

These approaches can greatly reduce the amount of fire protection required, and in some cases show that no applied protection is needed. Their main limitation is that they are based on the behaviour of isolated simply supported members, with limiting temperatures derived from standard ISO834 (International Organisation for Standardisation 1985) tests. In real buildings structural elements form part of a continuous assembly, and building fires often remain localised, with the fire-affected structure receiving significant restraint from cooler areas surrounding it. The real behaviour of these structural elements can therefore be very different from that indicated by standard furnace tests.

In 1995-96 six large fire tests were carried out on a full-scale composite building at the BRE Fire Research Laboratory at Cardington (Swinden Technology Centre 1999). The tests made it clear that unprotected steel members can have significantly greater fire resistance within real multi-storey buildings than when they are tested as isolated members. This appeared to be due to interaction between the heated members within the fire compartment, the concrete floor slabs and the adjacent steel frame structure. The most significant qualitative observation from the fire test results was that in none of the six was there any indication of run-away failure, which is the eventual outcome of all isolated member tests in which temperatures are progressively increased. This is particularly remarkable since in some cases unprotected steel beam temperatures reached over 1000°C, when the steel strength had reduced by over 95%; deflections always exceeded *span/30* and in some cases exceeded the usual limit of *span/20*. Tensile membrane action in the concrete floor slabs may have played an important role in preventing run-away failure of the structure during the fire tests, especially when deflections had become very large.

In practice fire compartments in multi-storey commercial buildings often extend over complete storeys. The cost of fire testing at such a scale is prohibitive, and it is therefore becoming increasingly important to have analytical methods available which can predict the behaviour of such structures when subjected to fire. The computer program *Vulcan* (Najjar and Burgess 1996; Bailey 1995; Huang *et al.* 1999a; Huang *et al.* 1999b; Huang *et al.* 2000a;) has been progressively developed in recent years at the University of Sheffield to carry out three-dimensional modelling of composite building behaviour in fire. As part of this study the 36m x 36m generic composite flooring system has been designed. A series of analyses has been performed, based on different patterns of fire protection to the downstand steel beams. The influence of the steel reinforcement on the structural behaviour has been investigated. In order to develop a better understanding of the interactions between the cool and hot zones of the composite flooring system a series of analyses have been carried out, based on different extents and locations of fire compartments.

The generic composite flooring system used in the studies is based on realistic loading conditions and layouts, which can test a full range of protection strategies. The behaviour is compared against the critical temperatures for 30 and 60 minutes' fire resistance, calculated according to BS5950 Part 8, according to which all beams require fire protection for these two fire resistance periods.

ANALYSIS OF COMPOSITE FLOORS WITH DIFFERENT FIRE PROTECTION REGIMES

In the 3-dimensional non-linear finite element procedure which is the theoretical basis of *Vulcan*, a composite steel-framed building is modelled as an assembly of finite beam-column, spring, shear connector and slab elements. It is assumed that the nodes of these different types of element are defined in a common reference plane, which is assumed to coincide with the mid-surface of the concrete slab element. Its location is fixed throughout the analysis. The beam-columns are represented by 2-noded line elements. The cross-section of each element is divided into a number of segments to allow considerable variation of the distributions of temperature, stress and strain through the cross-section. Both geometric and material non-linearities are included. To represent the characteristics of steel-to-steel connections in a frame, a 2-noded spring element of zero length, with the same nodal degrees of freedom as a beam-column element, is used (Bailey 1995).

The interaction of steel beams and concrete slabs within a composite floor assemblage is represented using a linking shear-connector element, which is two-noded and has zero length; it employs three translational and two rotational degrees of freedom at each node (Huang *et al.* 1999b). In order to model the composite slabs including their ribbed lower portion, a modified layered orthotropic slab element has been developed. This element is based on the previously developed formulation (Huang *et al.* 1999a), in which the slab elements are modelled using a layered plate element based on Mindlin/Reissner theory. Each slab layer can have different temperature and material properties, which may be associated with thermal degradation. An effective stiffness model has been incorporated into the layered procedure to take account of the orthotropic properties of composite slabs. A maximum-strain failure criterion has been adopted. A smeared model has been used in calculating element properties after cracking or crushing has been identified at any Gauss point. After the initiation of cracking in a single direction, concrete is treated as an orthotropic material with principal axes parallel and perpendicular to the cracking direction. Upon further loading of singly cracked concrete, if the tensile strain in the direction parallel to the first set of smeared cracks is greater than the maximum tensile strain then a second set of cracks forms. After compressive crushing, concrete is assumed to lose all stiffness. The uniaxial properties of concrete and reinforcing steel at elevated temperatures, specified in EC4 (European Committee for Standardisation 1992), were adopted in this model. Full details of the modified layered procedure used are given in Huang *et al.* (2000a).

Recently the authors have further extended the layered procedure mentioned above to include geometric non-linearity in the modelling of slabs (Huang *et al.* 2000b). A quadrilateral 9-noded higher-order isoparametric element developed by Bathe (1996) is used in place of the previous 4-noded geometrically linear element, and a Total Lagrangian approach is adopted. In this non-linear layered procedure all previous developments in the modelling of material non-linearity are retained, including the effective stiffness modelling of ribbed composite slabs.

This study is based on a composite 36m x 36m floor structure comprising 4 bays 9m x 9m in each direction (Figure 1), subject to a whole-storey fire. All primary and secondary beams have been standardised respectively as 533x210x92UB and 356x127x39UB sections. A ribbed lightweight concrete slab of 130mm total depth is used, acting compositely with PMF CF70 profiled metal decking. The characteristic dead and imposed loads are assumed to be 4.08kN/m^2 and 2.5kN/m^2 respectively. From BS 5950: Part 8, the partial safety factors in fire are 1.0 for dead loads and 0.8 for non-permanent imposed loads, giving a total design load of 6.1kN/m^2 at the fire limit state. This loading is used throughout the paper, and represents load ratios of 0.42 for secondary beams and 0.41 for primary composite beams if S275 steel and C35 concrete are assumed.

In order to investigate the extent to which fire protection of the steel beams may be reduced as a result of the beneficial influence of the slab, three different protection regimes have been considered:

- **Protection Regime I.** All beams on the main gridlines are protected but other secondary beams are unprotected (Figure1).
- **Protection Regime II.** Similar to I, but secondary beams on Gridlines 2 and 4 are also left unprotected.
- **Protection Regime III.** All internal secondary beams are left unprotected, so that, compared to II, the protection of secondary beams on Gridline 3 has also been removed.

The temperature distributions in the unprotected beams and the concrete slab were assumed to follow the patterns indicated in the Cardington tests (Bann *et al.* 1995). This was represented by considering the cross-section as a number of zones - bottom flange, web and top flange of the steel beams and ten layers through the concrete slab, the temperature in each being taken from the Cardington test data. The temperatures in the protected beams were assumed to be 50% of those of the unprotected beams.

All beams were assumed to be simply supported. To save computing time, advantage was taken of symmetry (see Figure 1), so that only a quarter of the floor system needed to be analysed. In order to demonstrate the effect of the slab reinforcement on the structural behaviour a number of different meshes, ranging from A142 to A393, were considered.

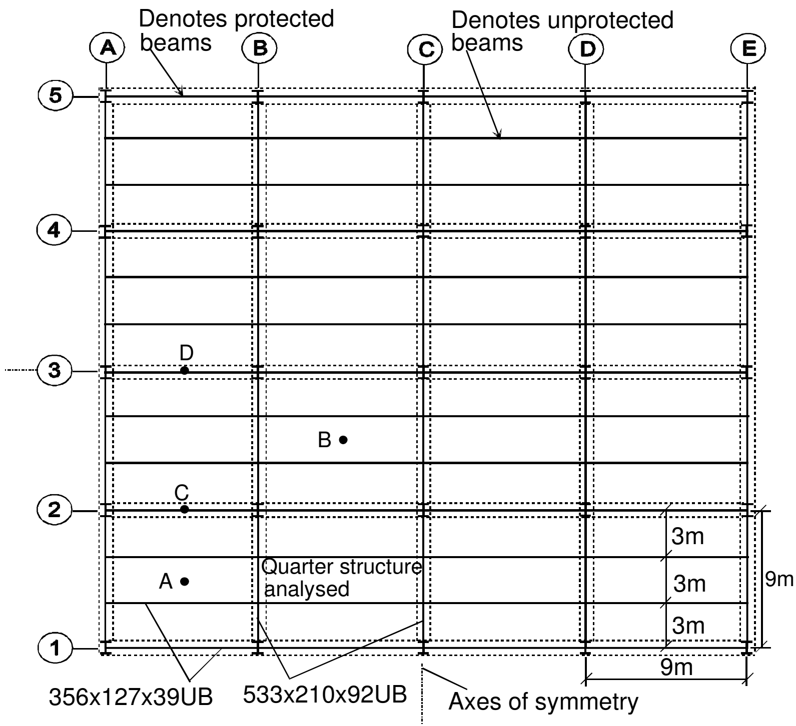

Figure 1: Composite floor layout assumed for Protection Regime I

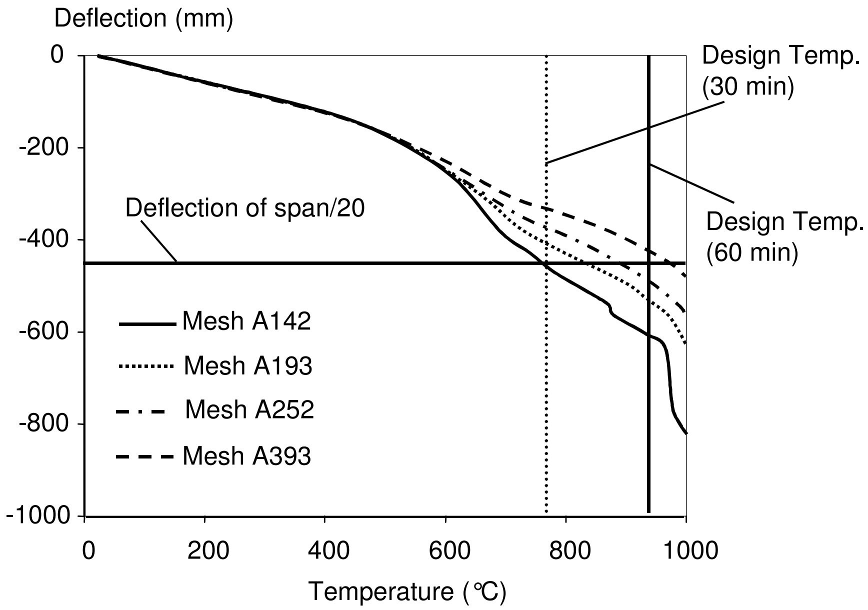

Figure 2: Protection Regime I: Predicted deflections at Position A with different reinforcement.

Protection Regime I

The vertical deflection at two key positions A and B on the frame analysed (see Figure 1) show that the influence of reinforcement is negligible up to about 500°C, but that beyond this it becomes increasingly significant (Figures 2 and 3). At these higher temperatures the steel beams have lost most of their original strength and stiffness, and support of the loads becomes increasingly the role of the concrete slab, with tensile membrane action being a key factor. For deflections greater than *span/30* the detailed output from *Vulcan* indicates a generally high level of tensile strains in the mid-regions of the concrete slabs. This suggests that most of the concrete is cracked, and therefore that it is the reinforcing steel which mainly resists the tensile membrane forces within the slab.

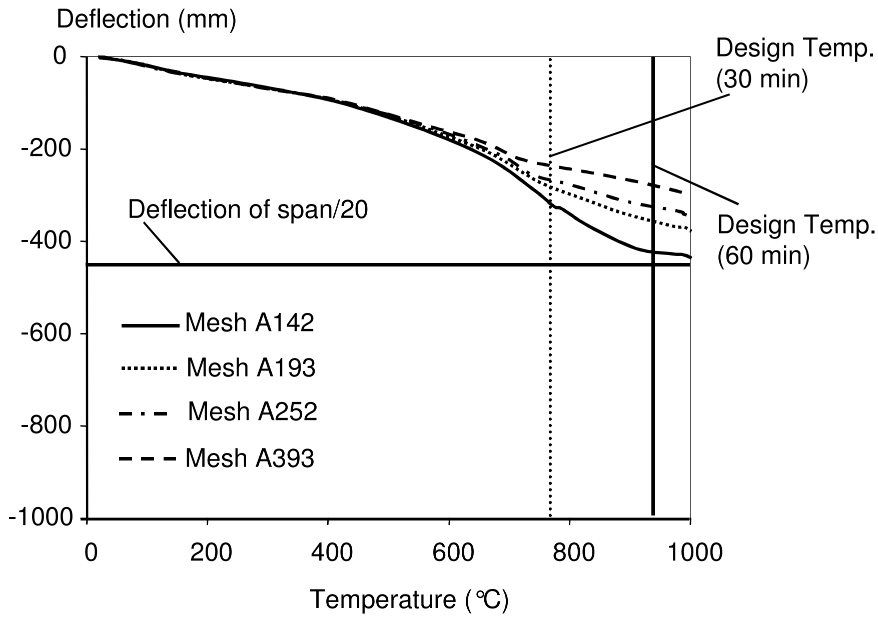

Figure 3: Protection Regime I: Predicted deflections at Position B with different reinforcement.

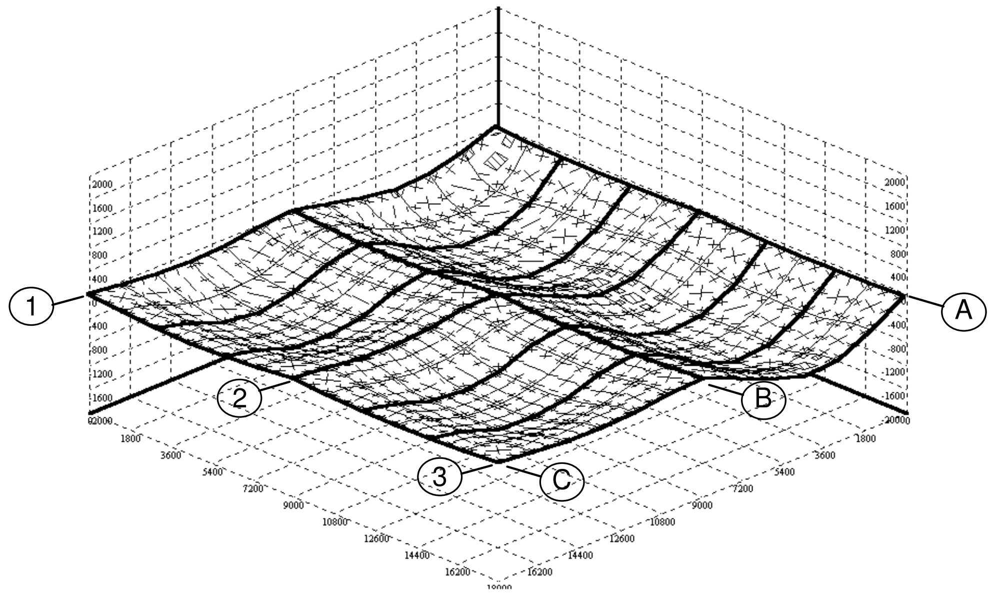

Figure 4: Protection Regime I: Deflection profiles at 1000°C, with cracking patterns of top layer of floor slab.

Comparing Figures 2 and 3 it is evident that the deflections of the internal central bay (Position B) are only about half of those in the corner bay (Position A). This demonstrates the influence of structural continuity in enhancing the fire resistance. Figure 2 also shows that for the A142 mesh the deflection increases suddenly at a key temperature of 965°C. This is due to a sudden increase in the rate of deflection of the protected beam on gridline 2 A-B as illustrated in Figure 4, which shows the deflection profiles at 1000°C.

Figure 5 shows a vector plot of the distribution of principal membrane tractions (forces per unit width) at the Gauss points of the slab elements at ambient temperature. In this plot the lengths of the vectors are proportional to their magnitudes; thin vector lines denote tension and thick lines denote compression. It can be seen that the slabs above the secondary and primary beams act very much according to the normal engineering assumption for the flanges of composite beams, being in compression parallel to the beam. This reduces somewhat in the areas mid-way between adjacent beams due to the well-known phenomenon of shear lag. In contrast, the membrane tractions within the slab at 900°C are shown in Figure 6.

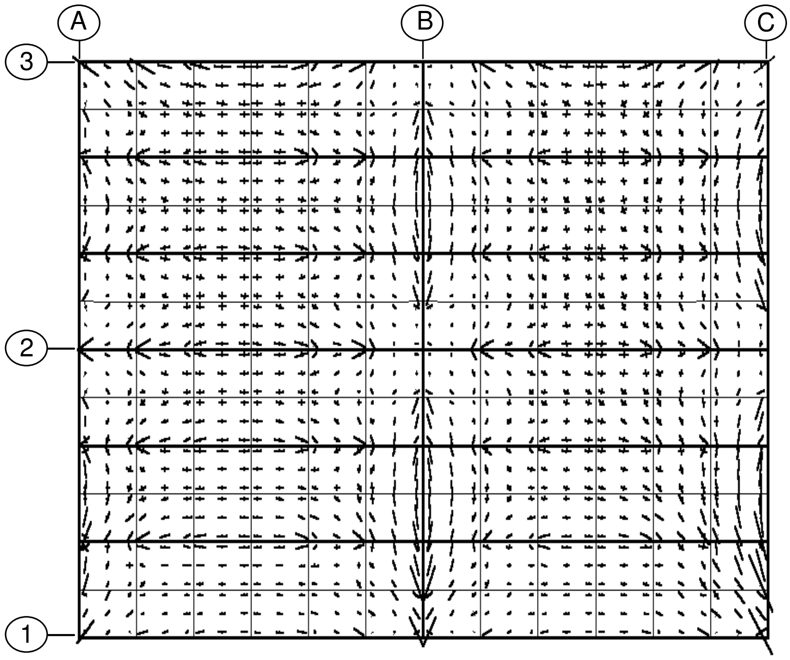

Figure 5: Distribution of principal membrane tractions at 20°C
(thick line = compression; thin line = tension).

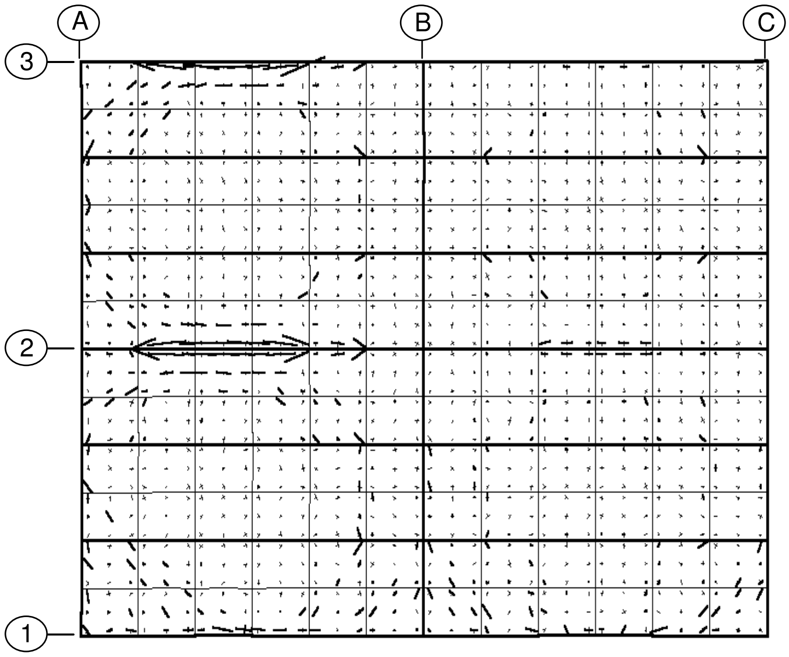

Figure 6: Protection Regime I: Distribution of principal membrane tractions at 900°C
(thick line = compression; thin line = tension).

Protection Regime I effectively optimises the potential for tensile membrane action in the concrete slab by providing, in its pattern of protected beams, edge-supported bays which are square in plan. In fact the increased rate of deflection at temperatures of about 965°C is associated more with the inability of the protected beams to maintain the required vertical support than with failure of the slab itself. This can be seen by comparing results (Figure 7) when the protected beams attain 50% of the unprotected beam temperatures with those when they are maintained at ambient temperature (termed 100% protection). In the latter case the deflections remain low, even when the unprotected structure has reached high temperatures.

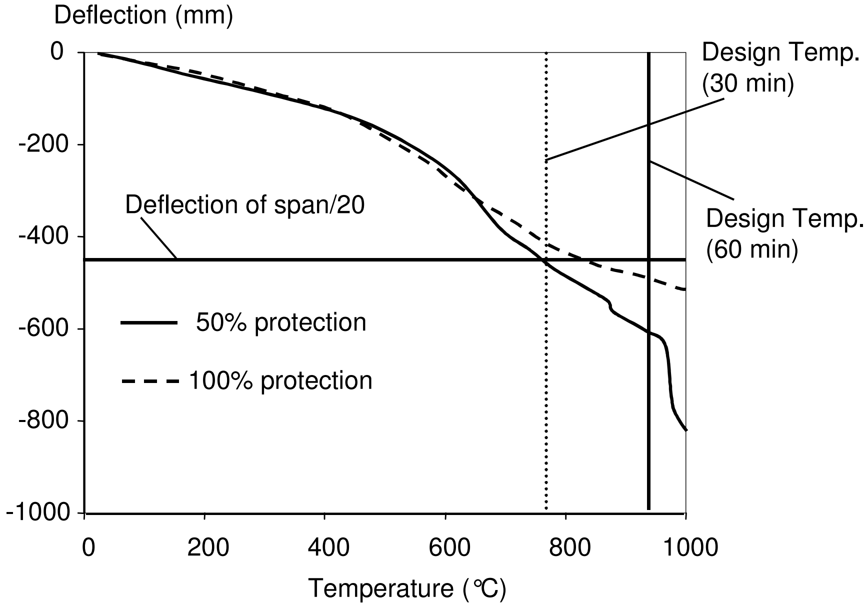

Figure 7: Protection Regime I: Predicted deflections at Position A with different degrees of protection for protected beams (mesh A142).

None of the more heavily reinforced cases is subject to run-away failure and the maximum deflection reached is about *span/15*. This high deflection level is consistent with that observed in the Cardington tests, suggesting that the deflection limit of *span/30* commonly used in association with structural fire testing may be too conservative in continuous structures.

The limiting temperatures of the 356x127x39UB secondary beams and 533x210x92UB primary beams were calculated according to BS5950: Part 8 as 674°C and 677°C, respectively. For a fire resistance period of 30min the design temperatures are 767°C and 739°C respectively and all beams would need fire protection. However, for A142 mesh the *Vulcan* analysis (Figure 2) shows that the structure reaches a deflection of 460mm, slightly higher than *span/20*, at about 770°C, and that the protection regime would therefore provide the required fire resistance. For a 60min fire resistance period the design temperatures increase to 938°C and 935°C for the secondary and primary beams respectively. For A393 mesh a deflection level of *span/20* (450mm) is reached at 970°C, and using this mesh with the current fire protection regime would therefore provide 60min fire resistance.

Protection Regime II

For this protection regime the maximum deflections were found to occur at position C, and the deflection history at this point is shown in Figure 8. For temperatures up to about 600°C the slab has very little influence. Between 600°C and 800°C there is some tensile action in the slab, but this is associated principally with catenary action rather than membrane action. This is because the pattern of vertical support provided by the protected beams results in rectangular rather than square bays. At high temperatures the strength of the protected beam on gridline 3 begins to reduce significantly, and this further compromises the ability of the slab to develop any membrane action. The catenary action is dependent on adequate anchorage, and this can only be provided by the external columns in this case. Consequently its effect is very limited, and at temperatures greater than 800°C these columns begin to pull in, reducing the capacity to develop catenary action, and floor deflections generally start to increase significantly. This is illustrated in the deformation pattern at 950°C (Figure 9).

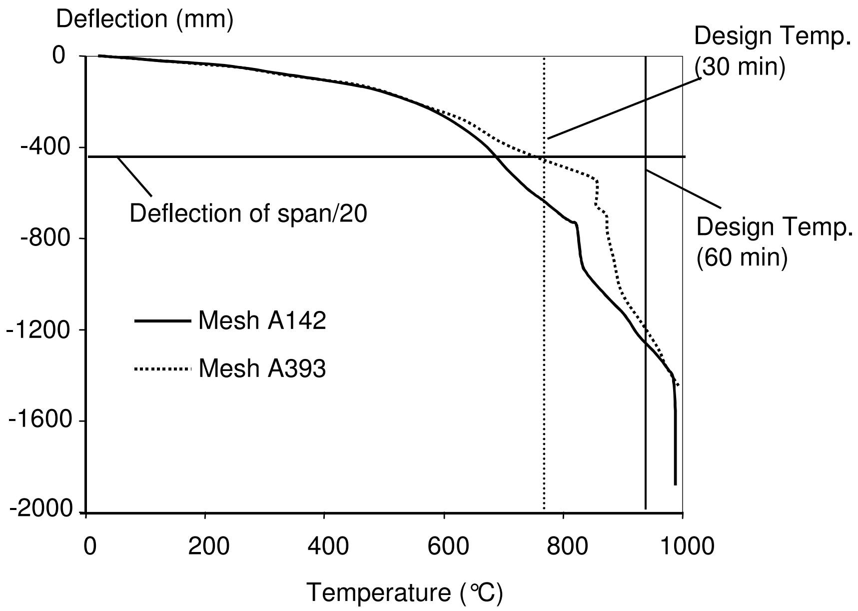

Figure 8: Protection Regime II: Predicted deflections at Position C with different areas of reinforcement.

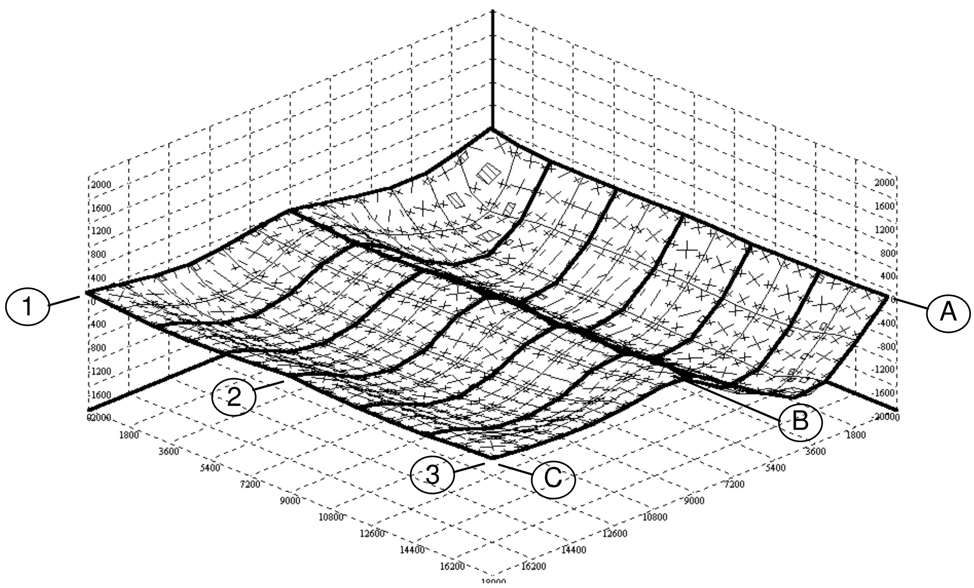

Figure 9: Protection Regime II: Deflection profiles at 950°C, with cracking patterns of top layer of floor slab.

It is clear from these analyses that the strength and stiffness of protected beams has a very important influence on the mobilisation of tensile membrane action in the concrete slabs. It is possible for a large amount of load to be redistributed from unprotected beams to protected beams. It is important to ensure that the protected beams can carry this load without undergoing high deflection, so that slabs are forced to deflect quite distinctly in double-curvature. In this case tensile membrane action is generated. If the protected beams deflect to the extent that the slab deflection is predominantly in single-curvature then high in-plane restraint is necessary to avoid run-away failure. This will not normally be present in edge bays where this restraint is due mainly to the sway stiffness of columns. According to the results from *Vulcan* modelling a deflection of *span/20* is reached at unprotected beam temperatures of 690°C and 770°C for A142 and A393 mesh respectively. Hence, if a fire resistance of 30min is required A393 mesh should be used, whilst 60min fire resistance cannot practically be realised with this fire protection regime.

Protection Regime III

In Protection Regime III it can be expected that the fire resistance will be much less than for Protection Regime I. The analyses were performed for two reinforcing meshes, A142 and A393. Figure 10 shows the vertical deflections at key position D (see Figure 1), for different meshes.

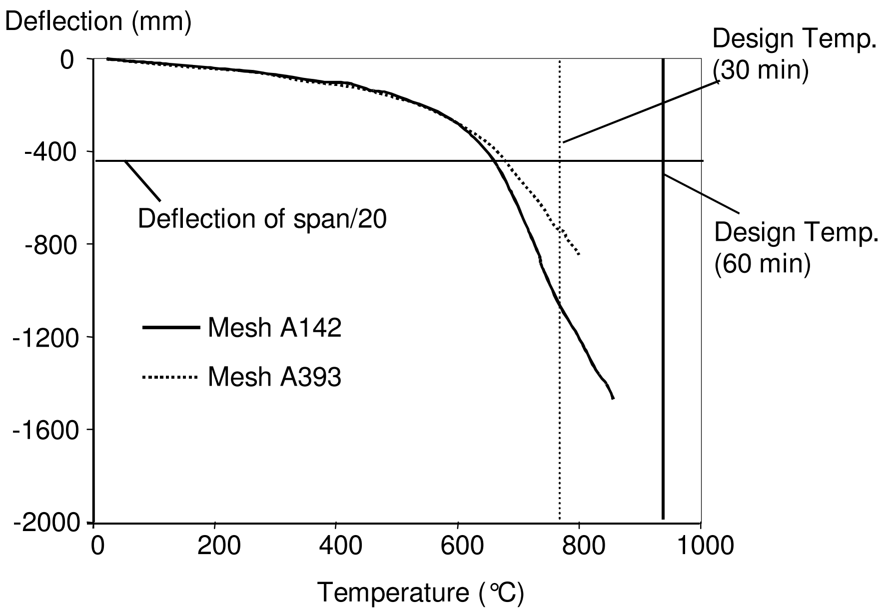

Figure 10: Protection Regime III: Predicted deflections at Position D with different areas of reinforcement.

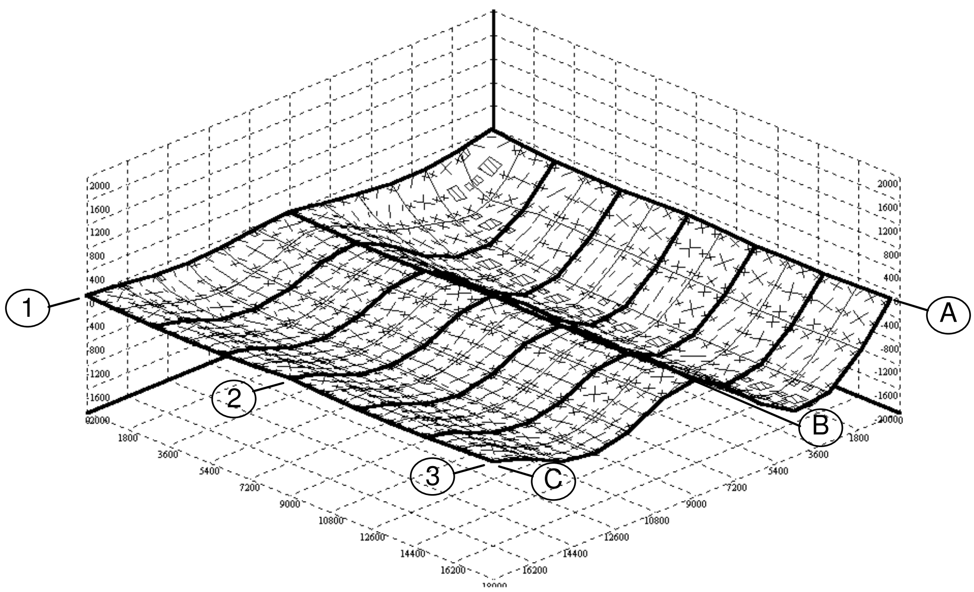

Figure 11: Protection Regime III: Deflection profiles at 800°C, with cracking patterns of top layer of floor slab.

From the figure it can be seen that when the temperature of the unprotected beams was above 700°C a runaway failure of the structure was initiated. The deflection profile of the composite floor system when the unprotected beams were at 800°C is shown in Figure 11 for A142 mesh. It is clear that the failure mode was predominantly single-curvature bending, with some pull-in of the edge beams. Because of this mainly single-curvature slab deformation the influence of tensile membrane action was minimised. From Figure 10 it can be seen that a deflection of *span/20* was reached at an unprotected beam temperature of 685°C for the A393 mesh case, and so 30 minutes' fire resistance cannot be achieved with this fire protection regime.

INFLUENCE OF DIFFERENT EXTENTS AND LOCATIONS OF FIRE COMPARTMENTS

In order to develop a better understanding of the interactions between the cool and hot zones of the composite flooring system a series of analyses have been carried out based on different extents and positions of localised fire compartments. In this analysis fire compartments in four different locations were modelled (see Figure 12), and A142 mesh is used in all slabs. Again the temperature distributions in the unprotected beams and the concrete slab within the fire compartment were assumed to follow the patterns indicated in the Cardington tests (Bann *et al.* 1995). The edge beams of the fire compartment were protected and the temperature of the protected beams was assumed to be 50% of that of the unprotected beams. In the following text the key temperature quoted in all figures refers to the temperature of the bottom flanges of all the unprotected beams.

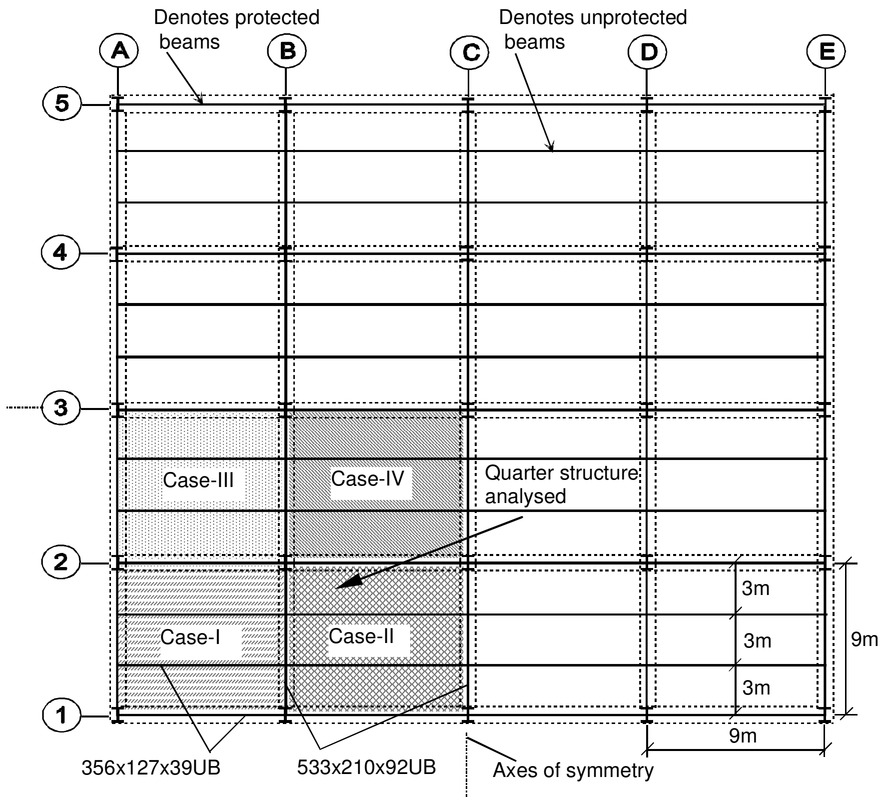

Figure 12: Composite floor layout, with different fire compartment positions marked.

Figure 13 plots the central deflection of the fire compartment against the key temperature for the four different cases, of which Case I and Case IV are the extremes. The influence of location of the fire compartments is clearly significant, especially when temperatures are high. The restraint from adjacent cool structure and continuity of floor slabs provide some benefits in increasing the fire resistance of the compartments. For a deflection level of *span*/20, the key temperature in Case IV is about 120°C higher than Case I. It is interesting to find that the deflection levels of the Cases II and III are very close to those of Cases IV and I, respectively. Hence, it is evident that the influence of restraint and continuity provided by cool floor areas at the edges of a fire compartment, which are perpendicular to the secondary beams is greater than that of areas at the edges perpendicular to primary beams. At a deflection level of *span*/20 the temperatures of unprotected secondary beams for Case I reach 785°C, which is slightly higher than the design temperatures of 767°C for a fire resistance period of 30min. In Case IV the structure reaches a deflection of 467mm, slightly higher than *span/20*, at about 940°C and this case would therefore provide 60 minutes' fire resistance, for which the design temperature is 938°C.

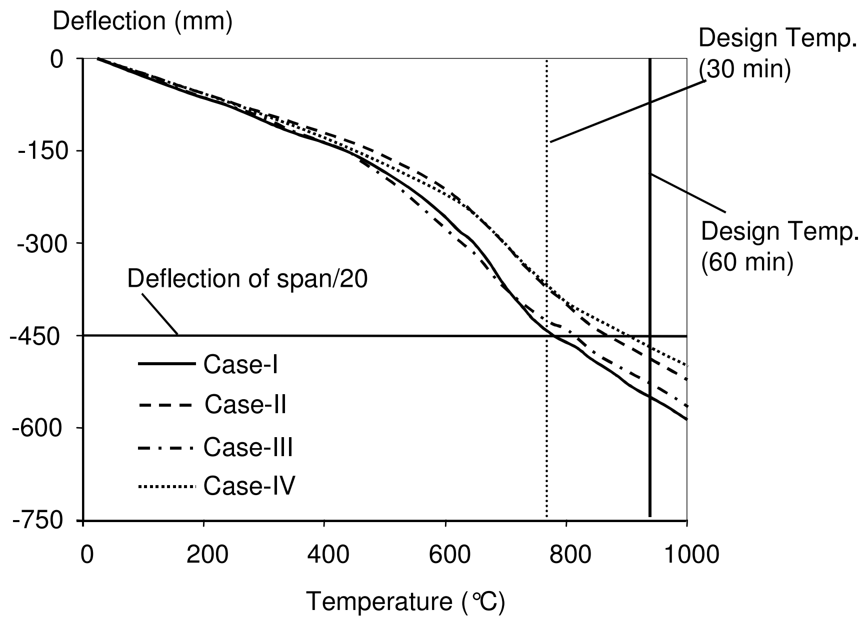

Figure 13: Comparison of predicted central deflections with different fire compartment positions.

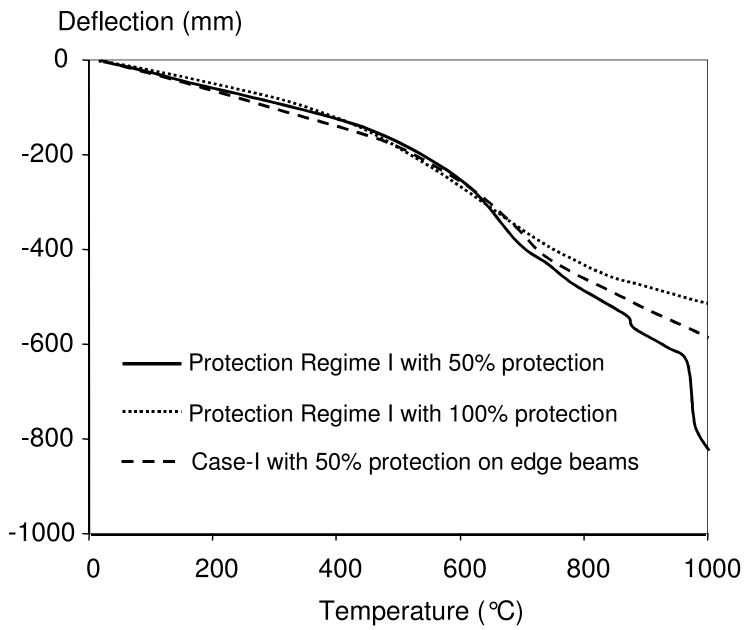

Figure 14: Comparison of predicted central deflections of corner bay with different fire protection to protected members.

In order to demonstrate the influence of cool floor areas on the corner bay fire compartment, a comparison is made between the localised Case I and whole-storey fires under Protection Regime I, in which protected beams are assumed to have either 50% or 100% protection. The results are plotted on

Figure 14 as the deflections at Position A (see Figure 1). It is clear that the influence of cool adjacent areas on the corner bay fire compartment is low. The differences are due to the different degrees of deformation of the protected beams, caused by the redistribution of loads in the whole-storey fire case. In other words, if the protected beams are strong enough, the structural behaviour of corner bays for these two fire cases is very similar. Figure 15 shows a comparison of the deflection at Position B (see Figure 1) between Case IV and whole floor heated under Protection Regime I, with 50% protection to protected beams. The effect of second-order forces due to restraint from the surrounding cool floor areas in Case IV can clearly be seen.

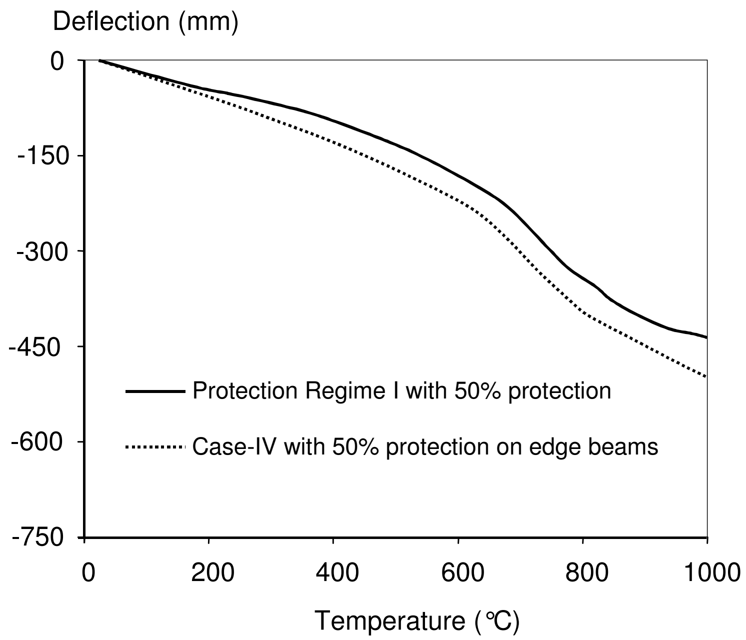

Figure 15: Comparison of predicted central deflections of central bay with different fires.

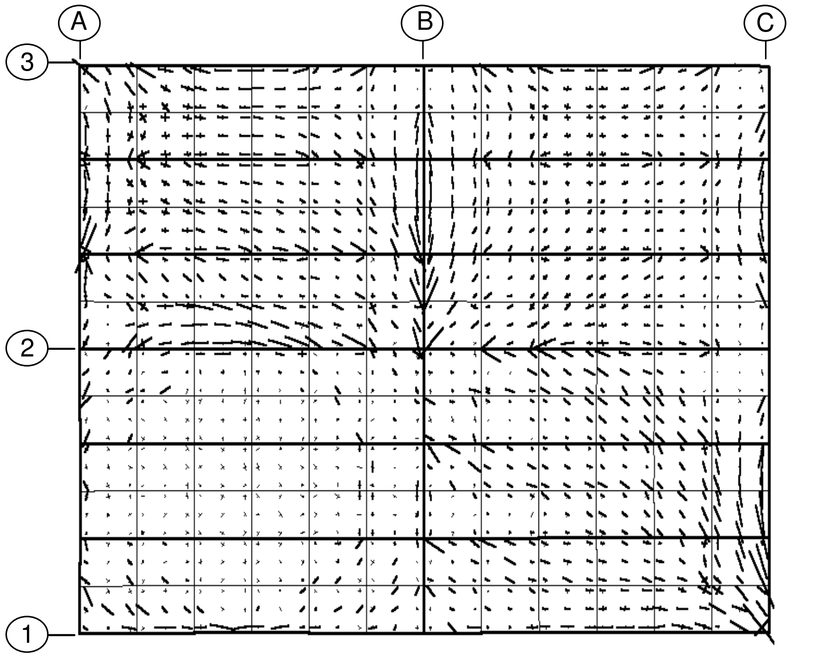

Figure 16: Case I: Distribution of the principal membrane tractions at 1000°C (thick line = compression; thin line = tension).

Figure 16 shows the distribution of the principal membrane tractions in the slab at 1000°C for Case I. In this corner bay fire compartment there is little possibility of in-plane restraint being provided by adjacent cool floor areas, and so forces in the corner bay floor slab need to be almost self-equilibrating in the horizontal plane. This means that the tensile membrane tractions in the central zone of the floor slab are balanced by the compression forces formed around its perimeter. The action is made possible by the presence of vertical support due to the protected edge beams, which forces the slab to deform in double-curvature and thus to generate the membrane traction field. The load-carrying capacity of the slabs is increased significantly due to this tensile membrane action, in which the anti-cracking reinforcement mesh is a key component. In contrast, distributions of the two principal membrane tractions in the slab at 1000°C for Case IV, which is in the central area of the composite floor and is subject to high restraint from surrounding cool floor areas, are shown in Figure 17. It can be seen that some tensile membrane forces are still generated within the centre of the fire compartment due to high deflection, and slabs deform in double curvature in this high-restraint case.

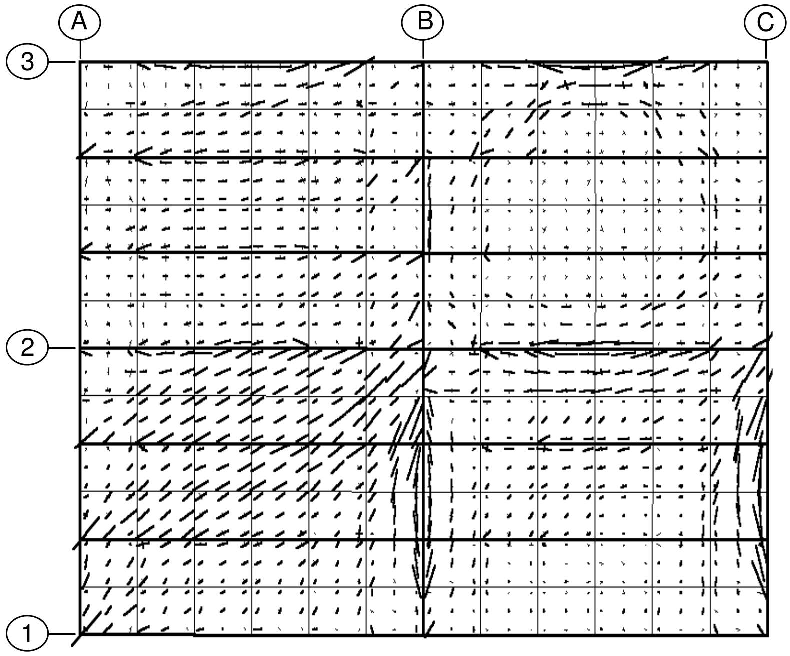

Figure 17: Case IV: Distribution of the principal membrane tractions at 1000°C (thick line = compression; thin line = tension).

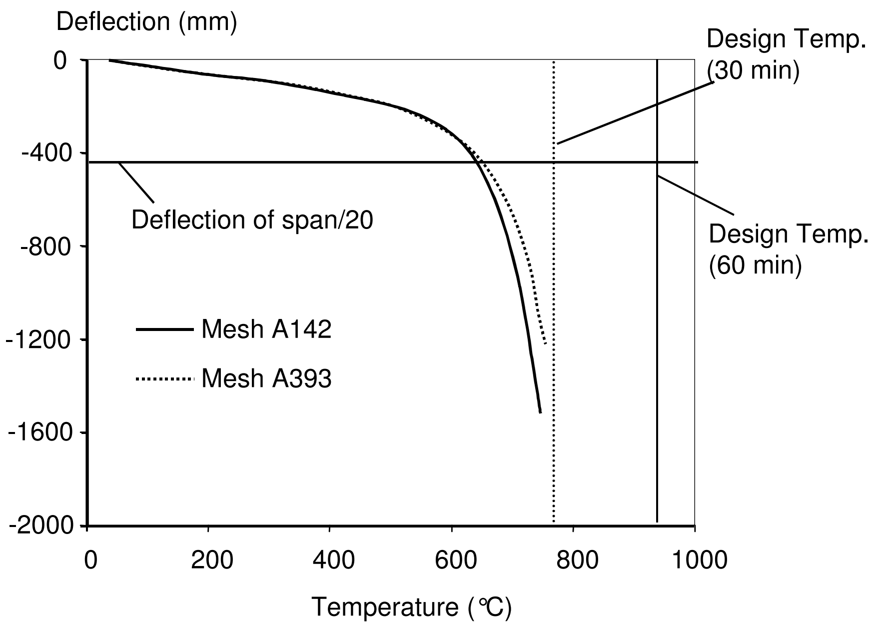

Figure 18: Predicted deflections with different areas of reinforcement at Position A.

The final case modelled, for the purpose of demonstration, is one in which the floor of a complete storey is heated with all beams left unprotected, and two reinforcing meshes are considered. The vertical deflection at Position A (see Figure 1) is shown in Figure 18, which indicates that the deflections start to run away as the temperature of the unprotected beams exceeds 650°C. Figure 19 shows the deformed profile of the floor system at 725°C for A142 mesh, and it is clear that the outer bays are behaving predominantly in one-way bending. It may be assumed that the internal bays, for which the continuity across gridlines helps to create a much greater degree of double curvature, will have higher fire resistance than is indicated by current design codes. However, at Position A an absolute deflection of *span/20* is achieved at 640°C, which is lower than the limiting temperature of 674°C given by BS5950 Part 8 (British Standards Institution 1990).

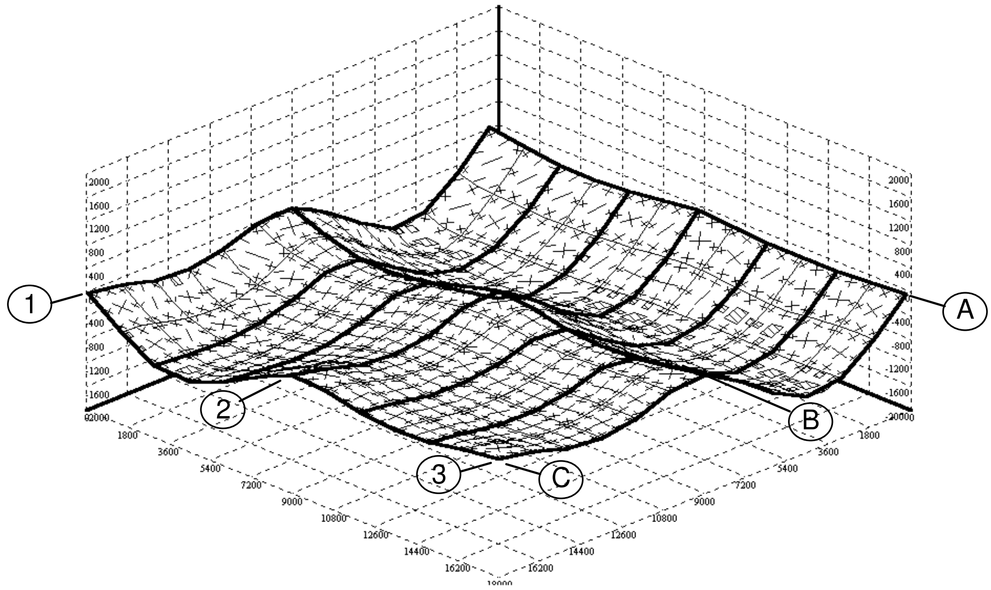

Figure 19: Deflection profiles at 725°C, with cracking patterns of top layer of floor slab.

If deflections are to be assessed for design purposes against the limits of *span/30* and *span/20* used in standard furnace testing of isolated members, it must be remembered that in such tests deflections are measured relative to the member ends. A proper assessment of deflections predicted by modelling such as this should really be based on a deflection measured relative to the end deflections of a composite beam or the edge deflections of a slab, and not on absolute values.

CONCLUSIONS

The generic composite flooring system used in the studies is based on realistic loading conditions and layouts which can test a range of protection strategies. A series of analyses has been performed, based on different patterns of fire protection to the downstand steel beams. It is evident that the presence or absence of tensile membrane action in the concrete slabs is a major influence on the ultimate integrity of the flooring system at high distortions. The extent to which tensile membrane action occurs depends very largely on the pattern of fire protection adopted for the steel downstand beams. In order to facilitate the mobilisation of tensile membrane action it is important to make sure that the concrete slab finally deforms in double curvature, and that it is incapable of producing folding mechanisms which do not need membrane straining. If this is not possible, then catenary action of slabs may occur, in which tension which is essentially uniaxial may be resisted by in-plane restraint from adjacent bays, beams and columns. However, this mechanism is much more likely ultimately to lead to run-away structural failures than is tensile membrane action. The ability of the slab reinforcement to sustain the tensile stresses caused at high temperatures and deflections is clearly a key factor in ensuring that fracture of slabs does not occur. This may necessitate either higher reinforcement ratios or placement of the mesh further from the heated surface of the concrete in order to increase its insulation.

A further series of analyses has been carried out based on different extents and locations of fire compartments. It is clear that the presence or absence of adjacent cool slab areas has a significant influence on the behaviour of the composite slab within the fire compartment. The restraint and continuity from this cool structure provide some benefits in increasing the fire resistance of the fire compartment. The behaviour has been compared against the limiting temperatures for 30 and 60 minutes' fire resistance, calculated according to BS5950 Part 8, according to which all beams require fire protection for these two fire resistance periods. From the *Vulcan* modelling it is indicated that a significant number of steel beams can be left unprotected for either fire resistance period. The analyses have also demonstrated the enhancements of fire resistance which can be achieved by increasing the area of the reinforcing mesh, or by using different fire protection strategies.

It is clear that current structural fire design codes, which are based on standard fire tests on isolated structural members, can not properly predict the structural behaviour of real buildings in fire conditions. Future developments in fire engineering design of composite structures should be based on an understanding of the interactions between different structural components within complete structural systems when subjected to fire.

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