STRATEGIES FOR FIRE PROTECTION OF LARGE COMPOSITE BUILDINGS

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ABSTRACT

In this study a large generic composite flooring system with a footprint of 36m x 36m has been designed. The frame is 4 bays wide and 4 bays deep, each bay having dimensions 9m x 9m. 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. 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. In order to optimise 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. 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 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 shown that a significant number of steel beams can be left unprotected for either fire resistance period. The analyses also demonstrate the enhancements of the fire resistance which can be achieved by increasing the size of the reinforcing mesh or 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 codes for the 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. Some comparisons are made with a new design method based on a simplified model of tensile membrane action in composite slabs, generally indicating that the method is appropriately conservative while suggesting that protected members on gridlines may need some extra protection when it is used.

INTRODUCTION

In the current British design standard BS5950 Part 8[1], and also in the draft Eurocode EC3 Part 1.2[2] 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, either inherent in the framing system used or 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[3] 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[4]. The tests made it clear that unprotected steel members have significantly greater fire resistance within real multi-storey buildings than when they are tested as isolated members. This appears 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 general 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 a complete storey. The cost of fire testing at such a scale is prohibitive, and it is therefore becoming increasingly important to have available analytical methods which can predict the behaviour of such structures when subjected to fire. The computer program *Vulcan*[5-9] has been progressively developed in recent years at the University of Sheffield to carry out three-dimensional modelling of composite building behaviour in fire. In this study 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, and various amounts of slab reinforcement. 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 limiting temperatures for 30 and 60 minutes' fire resistance, calculated according to BS5950 Part 8[1], according to which all beams require fire protection for these two fire resistance periods. The results are also used to check the applicability and conservatism of a recently published design approach[10, 11] which takes account of tensile membrane action in the floor slabs in a simplified manner. Space does not permit a proper re-statement of this method here. Briefly, however, it calculates an enhancement to the normal yield-line bending strength of an undeflected slab on the assumption that large deflection takes place using the yield-line configuration. It is further assumed that the slab yields simultaneously in pure tension across the whole of its shorter centre-line, and that fracture takes place according to a limiting average-strain criterion.

THEORETICAL BASIS OF THE PROGRAM

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. The reference plane 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[5,6].

The interaction of steel beams and concrete slabs within composite steel-framed buildings 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. The shear-connector element permits the modelling of full, partial and zero interaction at the interface between the concrete slab and the steel beam[8]. 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[7], in which the slab elements are modelled using a layered plate element based on Mindlin/Reissner theory and each 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 is 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 crushing, concrete is assumed to lose all stiffness. The uniaxial properties of concrete and reinforcing steel at elevated temperatures specified in EC4[12] were adopted in this model. Full details of the modified layered procedure used are given in reference 9.

Recently the authors have further extended the layered procedure mentioned above to include geometric non-linearity in their modelling of reinforced concrete slabs in fire[13]. A quadrilateral 9-noded higher-order isoparametric element developed by Bathe[14] 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.

STUDIES OF A GENERIC BUILDING STRUCTURE

This study is based on a composite 36m x 36m floor structure comprising 4 bays 9m x 9m in each direction (Fig. 1). All primary and secondary beams have been standardised as 533x210x92UB and 356x127x39UB sections respectively. A ribbed lightweight concrete slab of 130mm total depth is used, which acts compositely with PMF CF 70 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. This corresponds to 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 and 0.41 for the secondary and main beams respectively if the Cardington material properties 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, two different protection regimes have been considered, in addition to the case in which all of the beams are unprotected. For Protection Regime I all beams on the main gridlines are protected and other secondary beams are unprotected (Fig. 1). Protection Regime II is similar but secondary beams on gridlines 2 and 4 are also unprotected. The temperature distributions in the unprotected beams and the concrete slab were assumed to follow the patterns indicated in the Cardington tests[15]. 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 in the concrete slab, the temperature in each being taken from the Cardington test data. The temperature of the protected beams is assumed to be 50% of that of the unprotected beams. All beams are assumed to be simply supported. To save computing time, advantage is taken of symmetry (see Fig. 1), and only a quarter of the floor system needs to be analysed.

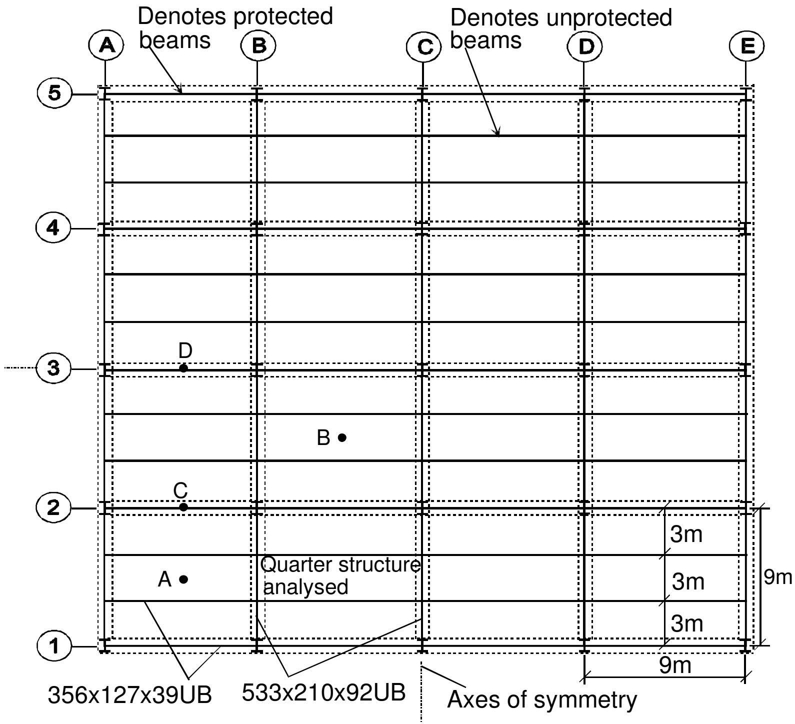

Fig. 1 Frame layout assumed for Protection Regime I

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.

Protection Regime I

The vertical deflection at two key positions A and B on the frame analysed (see Fig. 1) show that the influence of reinforcement is negligible up to about 500°C, but beyond this it becomes increasingly significant (Figs 2 and 3). At these higher temperatures the steel beams have lost most of their original strength and stiffness, and the loads become increasingly supported by the concrete slab, with tensile membrane action a key factor. For deflections greater than *span/30* the detailed output from *Vulcan* indicates a high level of tensile strains generally 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 makes the main contribution to the tensile membrane forces within the slab.

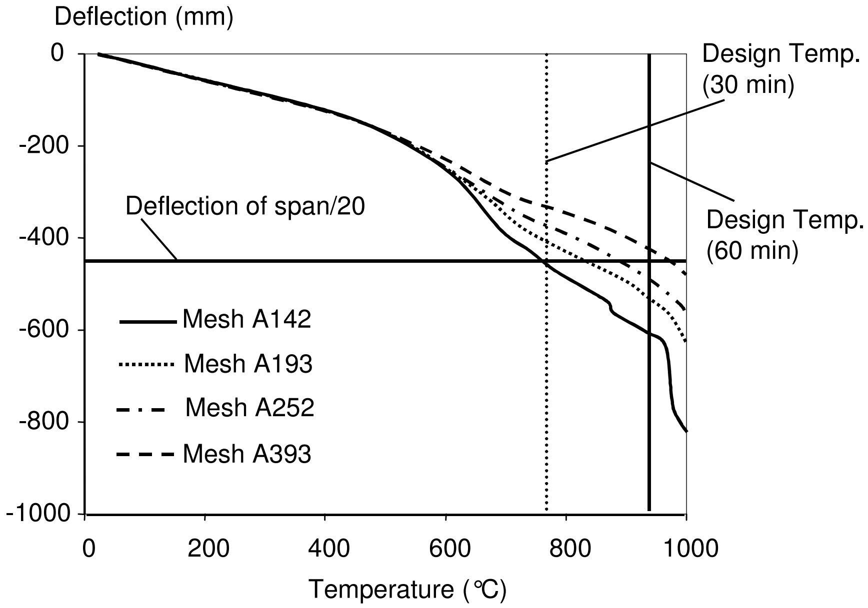

Fig. 2 Protection Regime I: Predicted deflections at Position A with different reinforcement.

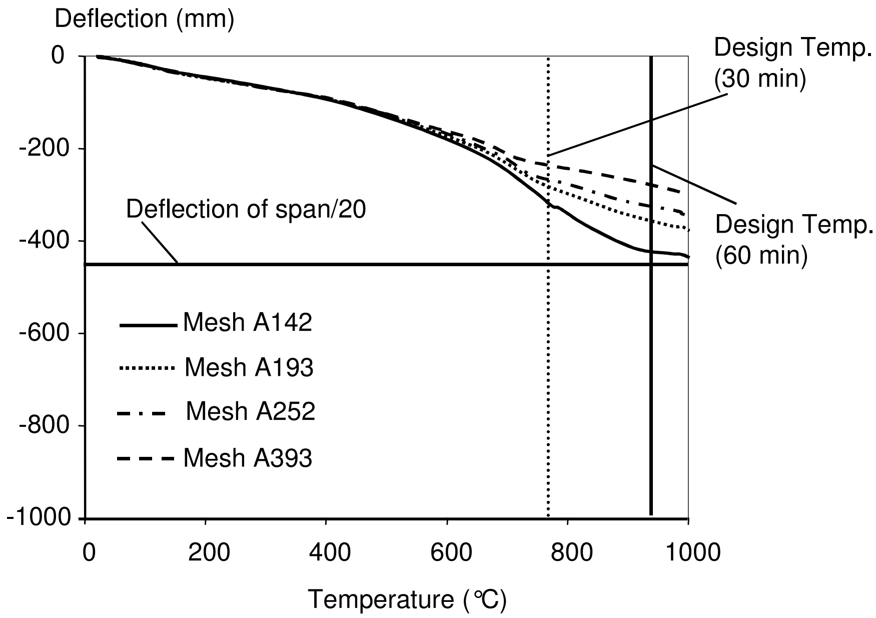

Fig. 3 Protection Regime I: Predicted deflections at Position B with different reinforcement.

Comparing Figs. 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. Fig. 2 also shows for the mesh A142 the deflection increases suddenly at temperatures 965°C. This is due to the sudden increase in the rate of deflection of the protected beam on grid line 2 A-B as indicated in Fig. 4, which shows the deflection profiles at 1000°C.

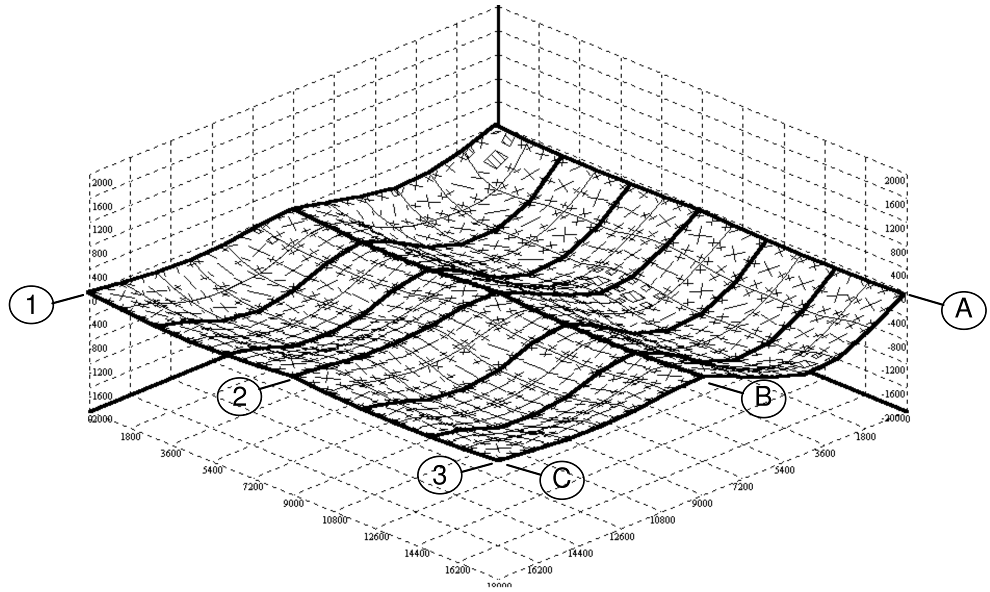

Fig. 4 Protection Regime I: Deflection profiles at 1000°C, with cracking patterns of top layer of floor slab.

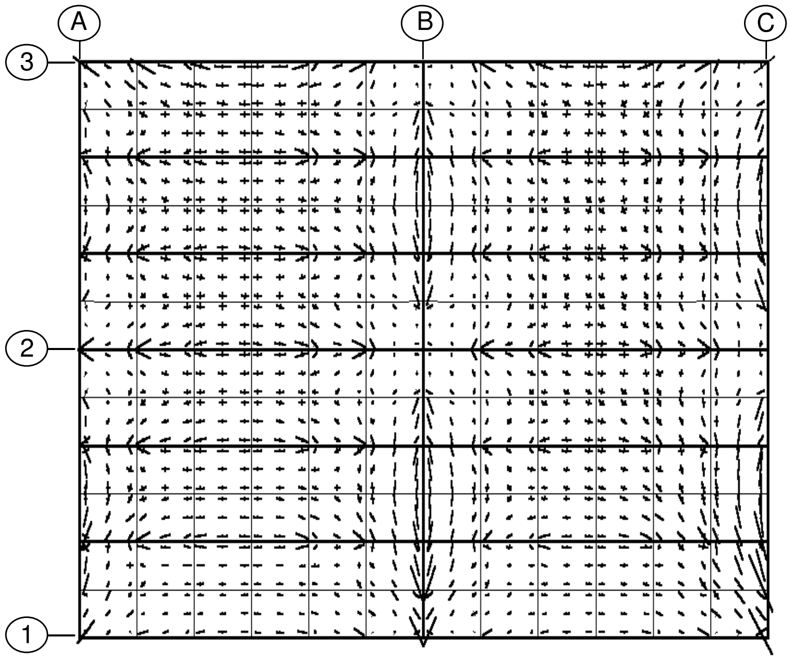

Fig. 5 Distribution of two principal membrane tractions at 20°C (thick line = compression; thin line = tension).

Fig. 5 shows the distributions of the two principal membrane tractions (forces per unit width) in the slab at ambient temperature. 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 Fig. 6.

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 deflections 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 (Fig. 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.

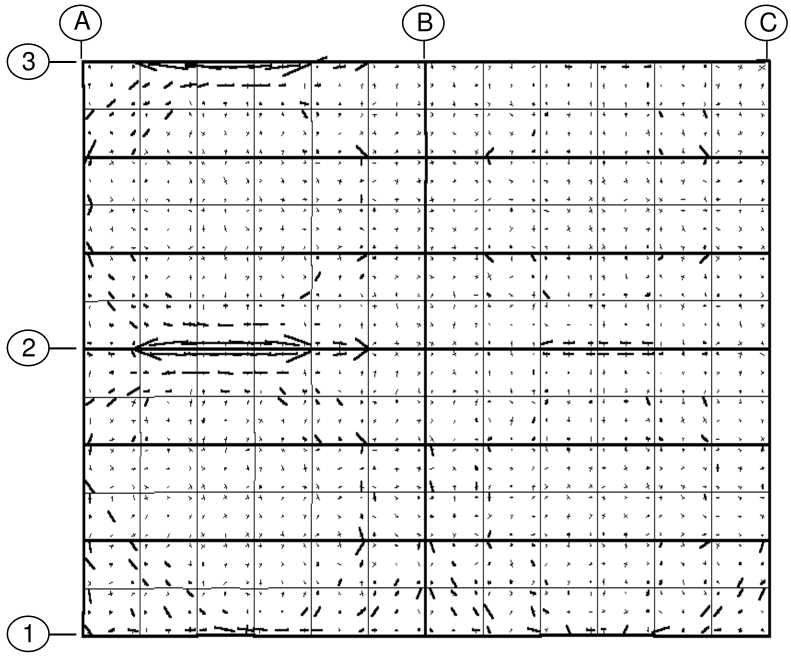

Fig. 6 Protection Regime I: Distribution of two principal membrane tractions at 900°C (thick line = compression; thin line = tension).

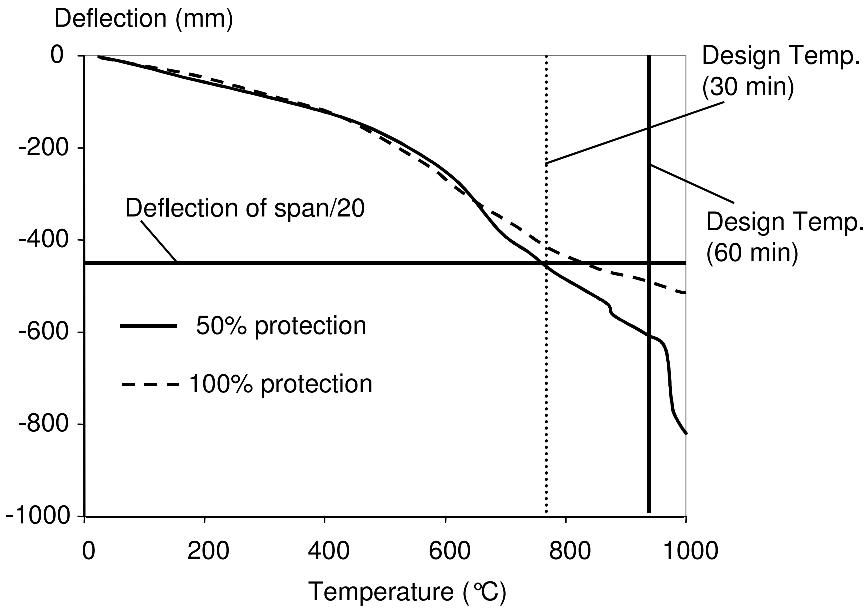

Fig. 7 Protection Regime I: Predicted deflections at Position A with different degrees of protection for protected beams (mesh A142).

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

The limiting temperatures of the secondary beams (356x127x39UB) and primary beams (533x210x92UB) 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 (Fig. 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.

The failure temperatures given by the new UK design calculation method are 676°C, 700°C, 731°C, and 878°C for meshes A142, A193, A252 and A393, respectively. These correspond reasonably closely to a deflection limit of *span/30* when compared with the deflection histories predicted by *Vulcan*, and are certainly conservative when compared with a limit of *span/20*. The design conclusions are therefore fully consistent with those stated above based on the *Vulcan* analysis.

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 Fig. 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 grid line 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 ability to develop catenary action and floor deflections generally start to increase significantly. This is illustrated in the deformation pattern at 950°C (Fig. 9).

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 the protected beams. It is important to ensure that the protected beams can carry these loads 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.

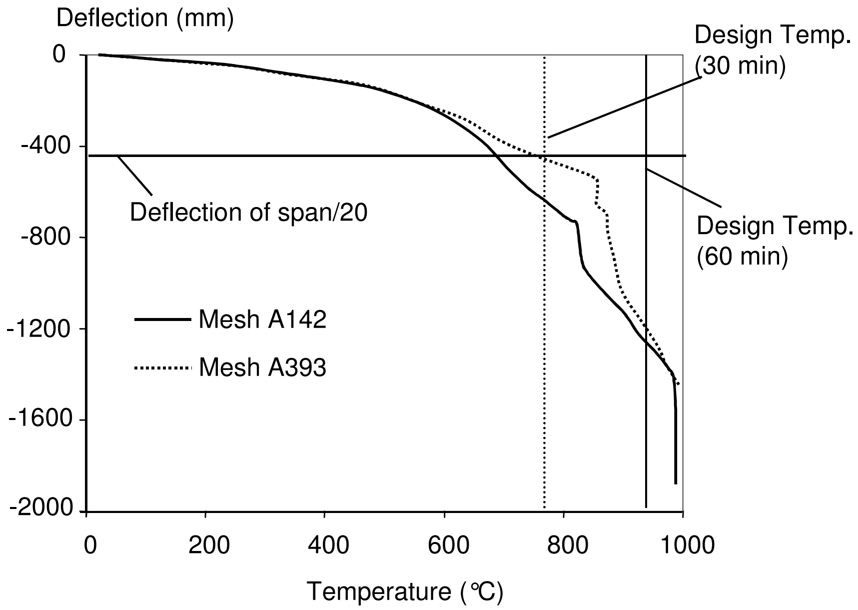

Fig. 8 Protection Regime II: Predicted deflections at Position C with different areas of reinforcement.

The failure temperatures according to the new design calculation method [10, 11] are 645°C and 691°C for meshes of A142 and A393, respectively. These again correspond reasonably closely to the deflection limit of *span/30* when compared with the deflection history predicted by *Vulcan*, and are conservative when compared with a limit of *span/20*. Hence, the design conclusions are consistent with those based on *Vulcan* analysis.

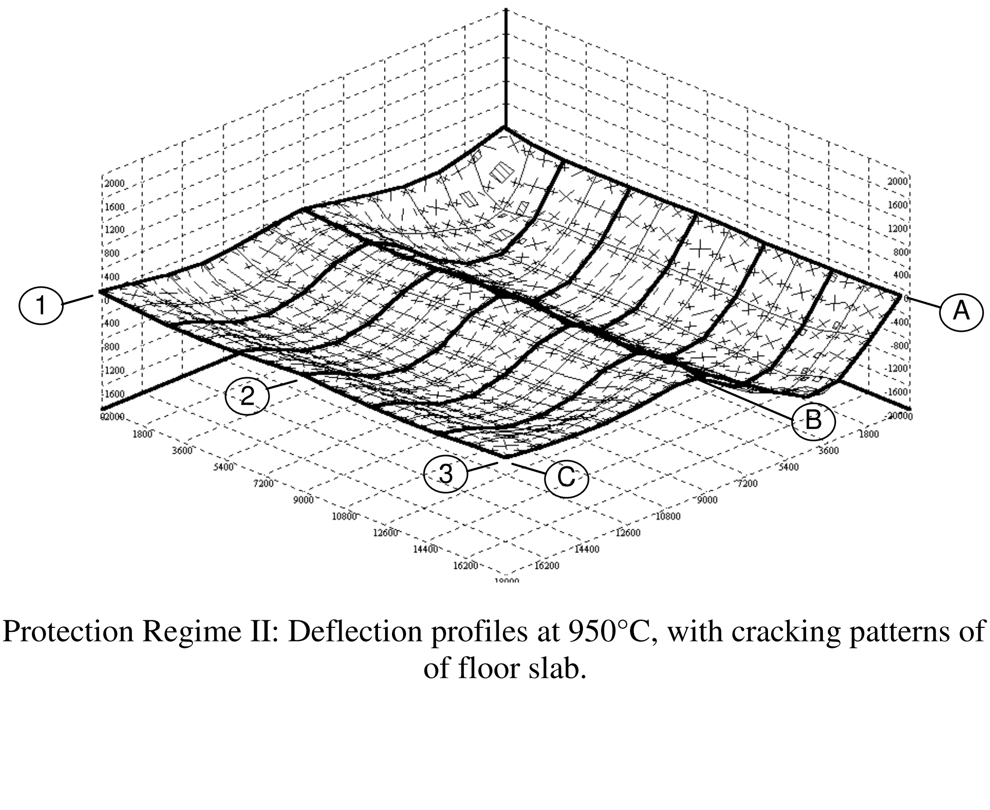

Fig. 9 Protection Regime II: Deflection profiles at 950°C, with cracking patterns of top layer of floor slab.

Protection Regime III: Unprotected Beams

In this case all beams are left unprotected, and two reinforcing meshes are considered. The BRE design procedure can not be applied to cases such as this, where no protected lines of support are provided. The vertical deflections at Position A (see Fig. 1) are shown in Fig 10, indicating that the deflections start to run away when the temperature of the unprotected beams exceeds 650°C. Fig. 11 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, in 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* has been achieved at 640°C, which is lower than the limiting temperature of 674°C given by BS5950 Part 8 [1].

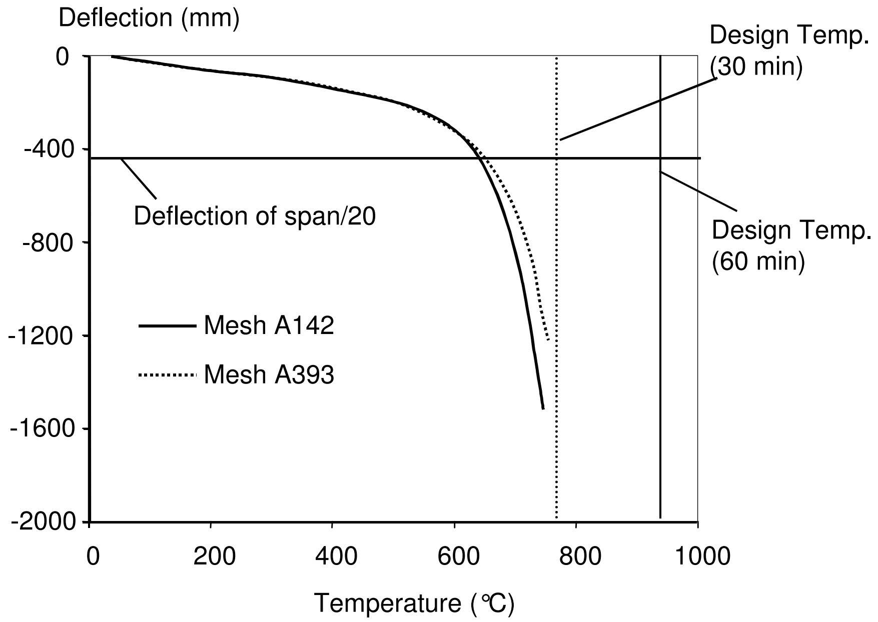

Fig. 10 Predicted deflections with different areas of reinforcement at Position A.

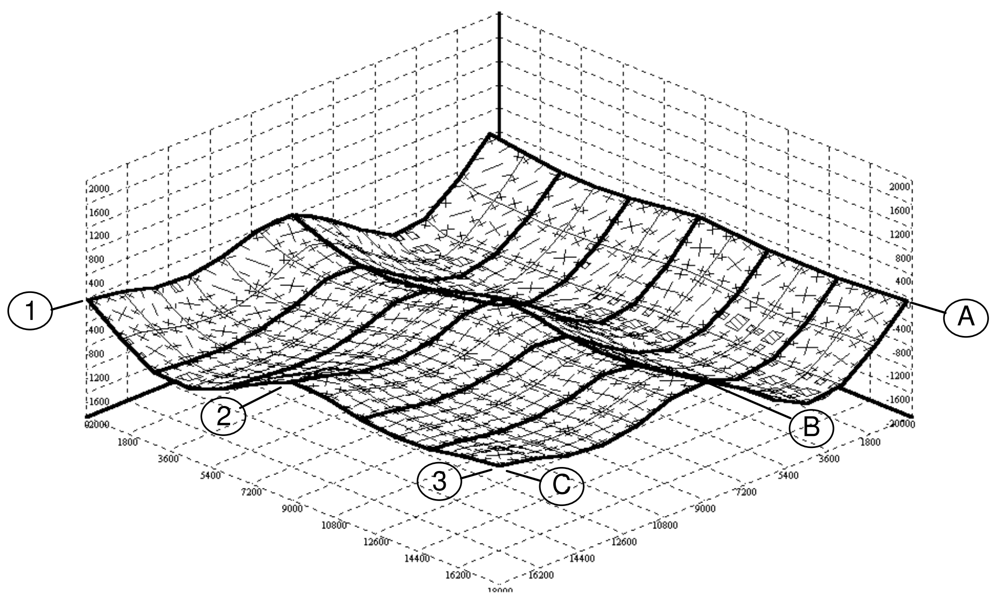

Fig. 11 Deflection profiles at 725°C, with cracking patterns of top layer of floor slab.

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 optimise 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.

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 both the *Vulcan* modelling and from the new BRE design method 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 the 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 codes for the 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. The BRE design method is the first to attempt this for composite floor systems, and is supported by the analyses conducted in this study.

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