BEHAVIOUR OF REINFORCED CONCRETE STRUCTURES IN FIRE

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ABSTRACT

In the past two decades, a significant amount of research has been conducted into the performance of composite steel-framed structures in fire. However, the same level of development has not taken place for other forms of construction. In terms of reinforced concrete construction, design is still based on simplistic methods which have been developed from standard fire tests that do not necessarily represent real building behaviour. This makes it very difficult, if not impossible, to determine the level of safety achieved in real concrete structures, or whether an appropriate level of safety could be achieved more efficiently. In this study detailed analyses of a reinforced concrete structure subject to a standard fire regime are carried out. The building is designed to Eurocode 2 and represents a commercial office building. In order to study the interactions between the cool and hot zones of the structure, a series of analyses has been carried out for different extents and positions of localised fire compartments. It is clear that adjacent cool structure provides considerable restraint and continuity, increasing the fire resistance of the structure within the fire compartment. Relatively small areas of tensile membrane force are formed within the concrete slabs, and large areas are subject to compressive membrane action during the fire. As a result the downstand concrete beams experience enhanced tension during the fire, especially in the early stages, which is mainly carried by their tensile reinforcement. It is therefore very important to keep the temperature of beam reinforcement within certain limits. Eventual structural collapse in the studies is always due to column failure, and it is clear that the performance of columns is vitally important to the survival of reinforced concrete buildings in fire.

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1. INTRODUCTION

The behaviour of structures exposed to fire is usually described in terms of the concept of fire resistance, which is the period of time under exposure to a standard fire time-temperature curve at which some prescribed form of limiting behaviour occurs. In performance-based design this limiting behaviour may be defined as real structural collapse or as a failure of integrity (which allows fire-spread to occur), but is more usually defined in terms of a deflection limit. Current design codes [1, 2] have taken a step towards full performance-based design by allowing designers to treat fire as one of the basic design limit states, taking account of:

- Non-uniform heating due to partial protection, which may be inherent in the framing system or specially applied,
- The level of loading in the fire limit state, using partial safety factors lower than those used for ultimate limit states, because of the relative improbability of such accidental conditions,
- Realistic stress-strain characteristics of materials at elevated temperatures.

The main limitation of these codified approaches is that they are based on the behaviour under test of isolated simply supported members, usually heated according to the standard ISO834 time-temperature curve[3]. In real buildings structural elements form part of a continuous assembly, and building fires often remain localised, with the fire-affected region of the 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 could have significantly greater fire resistance within real multi-storey buildings than when tested as isolated members. This was undoubtedly due to interaction between the heated members within the fire compartment, the concrete floor slabs (both heated and unheated) and the adjacent composite frame structure. If such interactions are to be used by designers in specifying fire protection strategies, as part of an integrated limit state design process, then this can not practically be based on testing because of the extremely high implicit costs. It is therefore becoming increasingly important that software models be developed to enable the behaviour of such structures under fire conditions to be predicted with sufficient accuracy.

A number of researchers have developed numerical modelling approaches to the behaviour of reinforced concrete structures in fire conditions. Ellingwood and Lin[5], and Huang and Platten[6], developed planar modelling software for reinforced concrete members in fire, and a simpler model has been developed by Lie and Celikkod[7] for the high-temperature analysis of circular reinforced concrete columns. In the major general-purpose finite element codes this kind of numerical modelling is often attempted, but the degradation of material properties tends to be simplified, and the finite element formulations used are often inappropriate for efficient set-up and analysis of concrete and composite buildings under fire attack.

The specialised finite element program *Vulcan* has been progressively developed over the past decade[8-11] at the University of Sheffield for three-dimensional modelling of the structural behaviour of composite and steel-framed buildings in fire. In this program a non-linear layered finite element procedure has been developed for predicting the structural response of reinforced concrete slabs subjected to fire [9, 10]. Also a more robust three-dimensional 3-noded beam-column element with general cross-section has been developed for modelling of steel and reinforced concrete frames in fire conditions[11]. These developments have provided a powerful tool to carry out 3D analysis of reinforced concrete structures in fire.

In this study a generic 37.5m x 37.5m normal-weight reinforced concrete structure comprising four floors has been considered, with realistic loading conditions and structural layout. In order to develop a better understanding of the interactions between the cool and hot zones of the structure, a series of analyses has been carried out for different extents and positions of localised fire compartments.

2. THEORETICAL BACKGROUND OF THE PROGRAM

In the 3D non-linear finite element procedure which is the theoretical basis of *Vulcan*, a reinforced concrete building is modelled as an assembly of finite beam-column and slab elements. It is assumed that the nodes of these different types of element are defined in a common reference plane, as shown in Fig. 1. 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 3-noded line elements. The model is based on a formulation proposed by Bathe[12] for geometrically non-linear modelling of elastic beams. The cross-section of the beam-column is divided into a matrix of segments, and each segment may have different material, temperature and mechanical properties. The complications of structural behaviour in fire conditions, such as thermal expansion, degradation of stress-strain curves, failure of concrete segments by cracking and crushing, and yielding of steel member or reinforcement segments, are included. The details of the formulations of beam-column elements and the constitutive modelling of concrete and steel at elevated temperatures have been presented previously[11].

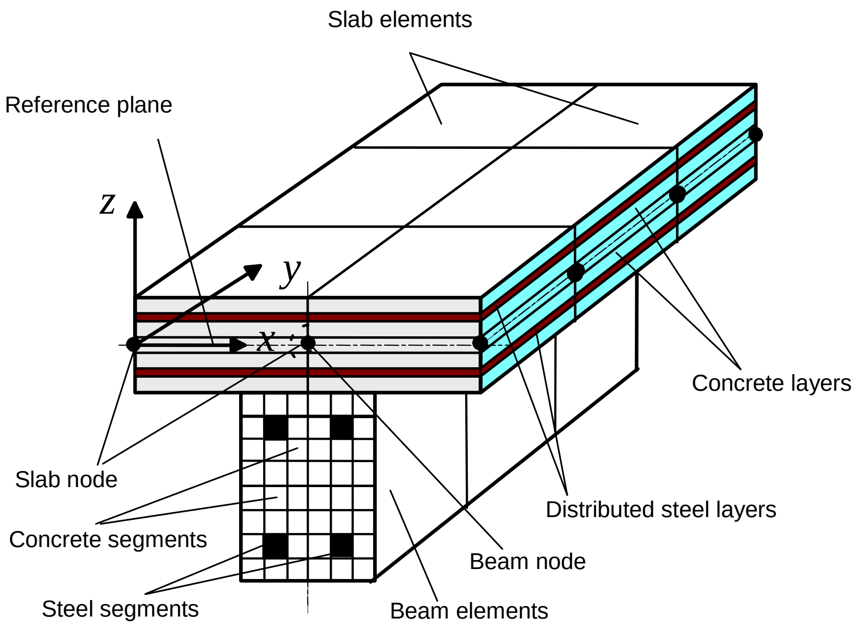

Fig. 1 – Division of reinforced concrete structure into beam and slab elements.

In this program a non-linear layered finite element procedure has been developed for predicting the structural response of reinforced concrete slabs subjected to fire [9, 10]. The procedure is based on Mindlin/Reissner (thick plate) theory, and both geometric and material non-linearities are taken into account. The slab elements are sub-divided into concrete and reinforcing steel layers to take into account temperature distributions through the thickness of slabs, thermal strains and material degradation for each layer, and failure layer-by-layer based on stress levels at Gauss points.

3. ANALYSIS OF REINFORCED CONCRETE STRUCTURE WITH WHOLE FLOOR HEATED

This study is based a generic 37.5m x 37.5m normal-weight reinforced concrete structure comprising four floors with 4.5m storey height and five 7.5m x 7.5m bays in each direction, which is subject to the ISO384 Standard Fire (see Fig. 2). The building is designed to Eurocode 2 [13] and BS8110 [14], and represents a commercial office building. The characteristic loads are assumed to be:

- Self-weight (assuming concrete density of 24kN/m^3): 7.5kN/m^2
- Raised floor: 0.5kN/m^2
- Ceiling and services: 0.5kN/m^2
- Partitions: 1.0kN/m^2
- Imposed load: 2.5kN/m^2

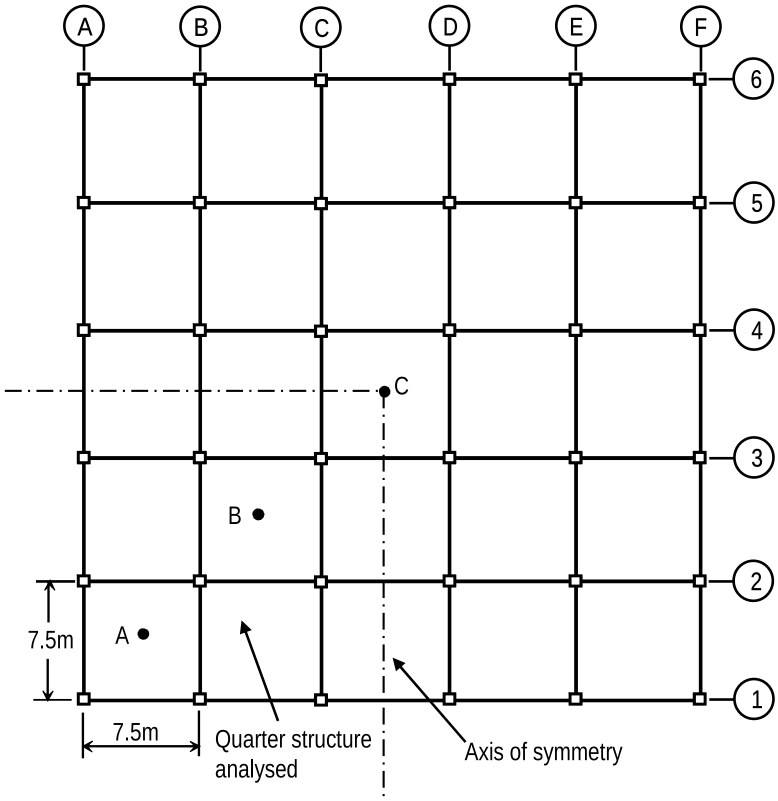

Fig. 2 - Concrete structure layout for whole ground floor heated by the ISO384 fire.

Therefore, at the fire limit state, the total design load on the structure is 10.75 kN/m^2 when the partial safety factor of 0.5 is applied to the imposed load. This loading is used throughout the example. The characteristic strength of concrete and reinforcing steel are assumed to be 45MPa and 460MPa, respectively. It is also assumed that two hours' fire resistance is required for the building. Therefore the nominal covers of beams, slabs and columns for the required fire resistance are 30mm, 25mm and 25mm respectively [14]. According to Eurocode 2 [13] each floor slab is nominally 250mm thick and designed as a flat slab. The dimensions of the cross-sections of beams and columns are 500mm x 350mm and 350mm x 350mm respectively. Fig. 3 shows the cross-section details of beams and columns.

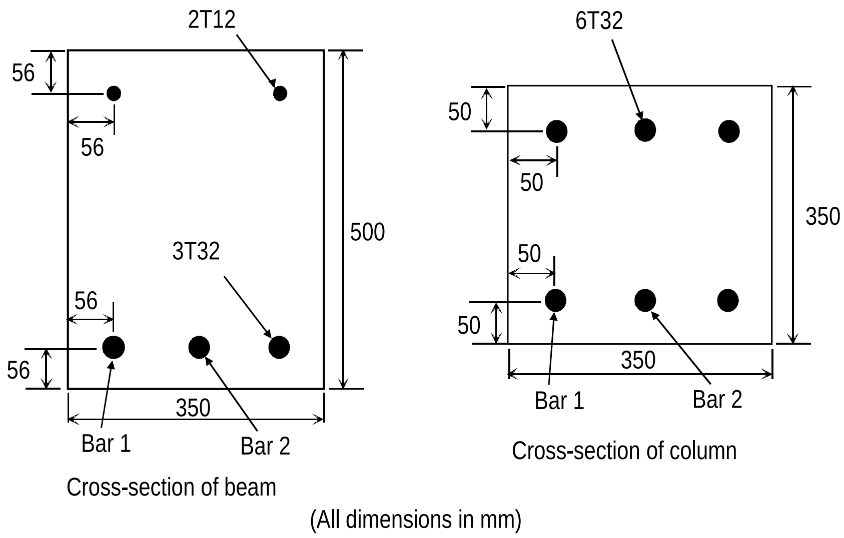

Fig. 3 - Cross-sectional details of beams and columns.

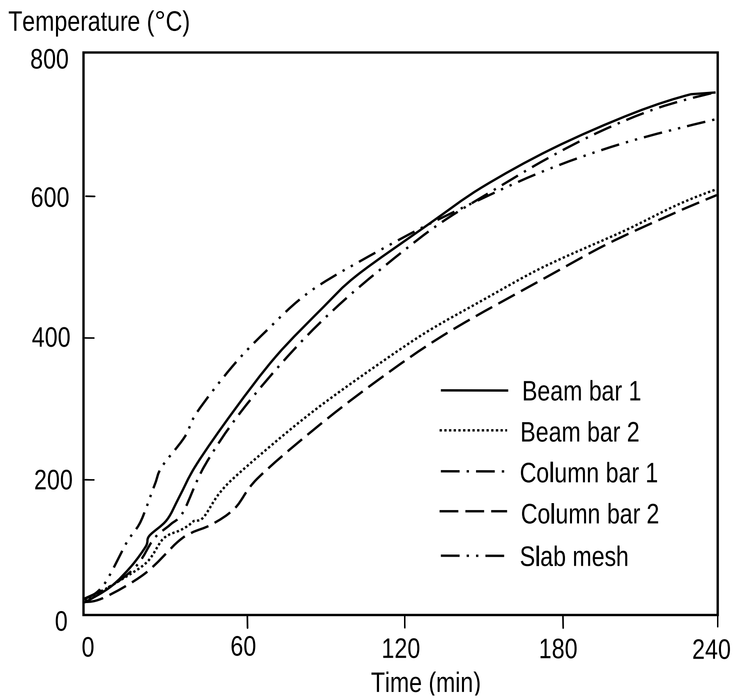

Fig. 4 - Predicted temperatures of main reinforcement for beams, columns and slabs.

In this case it is assumed that the whole ground floor of the building is engulfed in fire. Because of the inherent symmetry of the case, only a quarter of the structure is modelled. The first step is to perform a thermal analysis using *Vulcan*. The temperature histories of the main reinforcing steel bars in the beam and column sections, and the reinforcing mesh in the slab are shown in Fig. 4. The maximum temperatures of reinforcement at 120min and 180min are about 530°C and 660°C respectively. It is obvious that the concrete covers provide very good thermal insulation to the reinforcement during the fire. In this thermal analysis it is assumed that no concrete spalling happens in the fire condition.

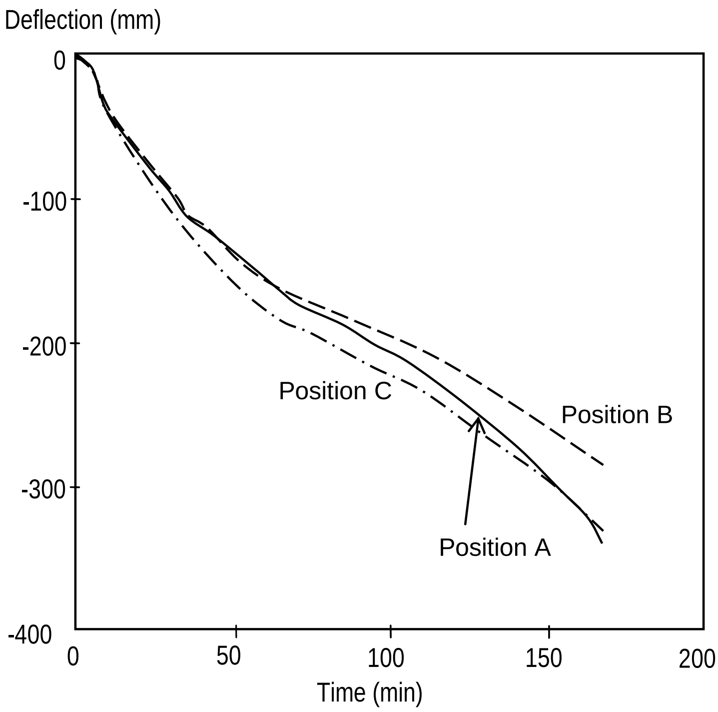

Fig. 5 - Predicted deflections at key positions of the concrete floor.

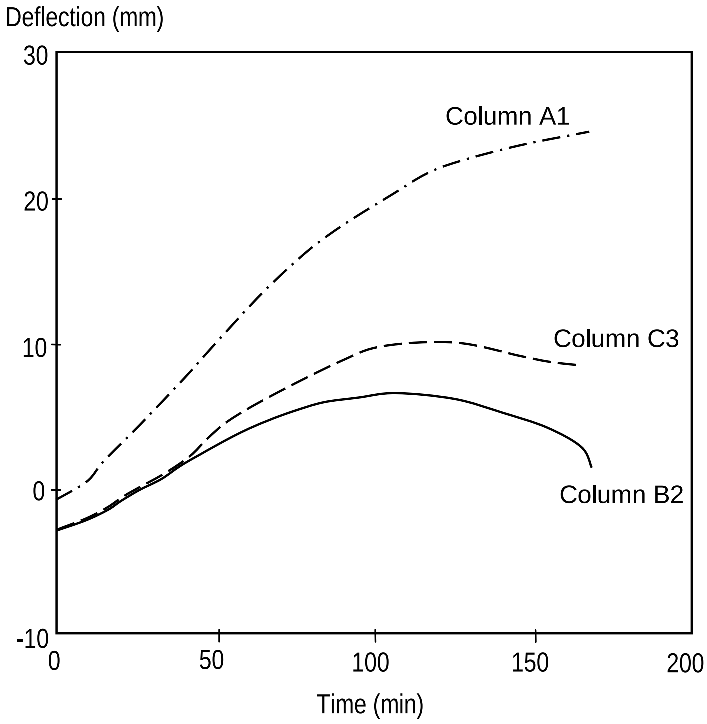

Fig. 6 - Predicted vertical deflections at the tops of three ground floor columns, A1, B2 and C3.

In other words, it is assumed that all the reinforced concrete cross-sections remain intact. The temperatures of the cross-sections of the members, generated by thermal analysis, are then used to carry out the structural analysis. The deflections of some key positions (see Fig. 2) within the structure analysed are presented in Fig. 5. It is evident that the maximum deflection of the floor slabs at 120min is about 250mm, which is *span/30*. Fig 6 shows the vertical deflections at the tops of three ground floor columns, A1, B2 and C3. It is evident that the columns initially extend upwards due to thermal expansion, then downwards after about 110min because of the reduction of the strength and stiffness of their concrete. The analysis was finally stopped due to buckling of Column B2. It is very clear that, as in composite structures, some fire protection of columns within reinforced concrete buildings is crucial for the extended survival of such structures in fire conditions. For reinforced concrete columns it is also essential to prevent spalling from the column faces in order to avoid direct exposure of the reinforcement to the fire.

Fig. 7 shows the deflection profile of the structure modelled at 150min. A vector plot of the distribution of principal membrane tractions (forces per unit width of slab) at the Gauss points of the slab elements at 150min is shown in Fig. 8. In this plot, the lengths of the vectors are proportional to their magnitudes; thin vector lines denote tension and thick lines denote compression. The figure shows repeated patterns clearly indicating tension fields in the central region of each floor panel, together with the peripheral compression 'rings' which are characteristic of tensile membrane action. It is obvious that the area in tension is rather smaller than the compression region. This is because the deflections are relatively small, less than *span/30*, so tensile membrane action is not very significant.

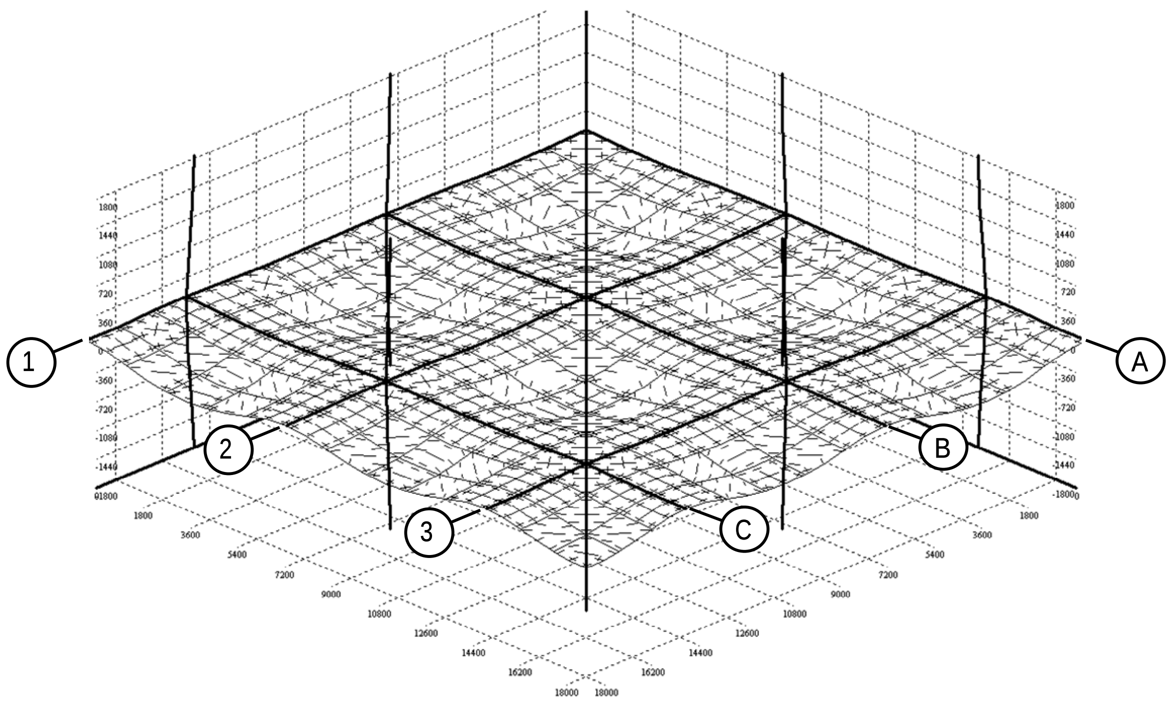

Fig. 7 - Deflection profiles at 150min, with cracking patterns of bottom layer of floor slab.

On detailed inspection of the compression and tension forces across the cut-lines (see Fig 8) it is clear that the compressive membrane forces are much larger than the tensile membrane forces. Hence, in order to maintain the equilibrium at this advanced stage of the fire, the reinforced concrete beams should probably carry tensile forces, since it is unlikely that the whole resultant of the slab forces is balanced by column shear. Fig. 9 shows the change with time of the axial forces of the concrete beams at three key positions shown in Fig 8. It can be seen that at ambient temperature the beams are all subject to resultant tension force. During the first 30min of the fire the tensile forces within the beams increase dramatically, to about three times the ambient-temperature value. Since concrete is very weak in tension the tensile forces within the beams are predominantly carried by the main reinforcing bars. It is evident from the figure that even after 150min the tensile forces of the beams are about twice their values at ambient temperature. Hence the possibility of tensile failure of the beam reinforcement in the initial stages of the fire is quite high, depending on the type of fire and dimensions of cross-sections of the beam and slabs.

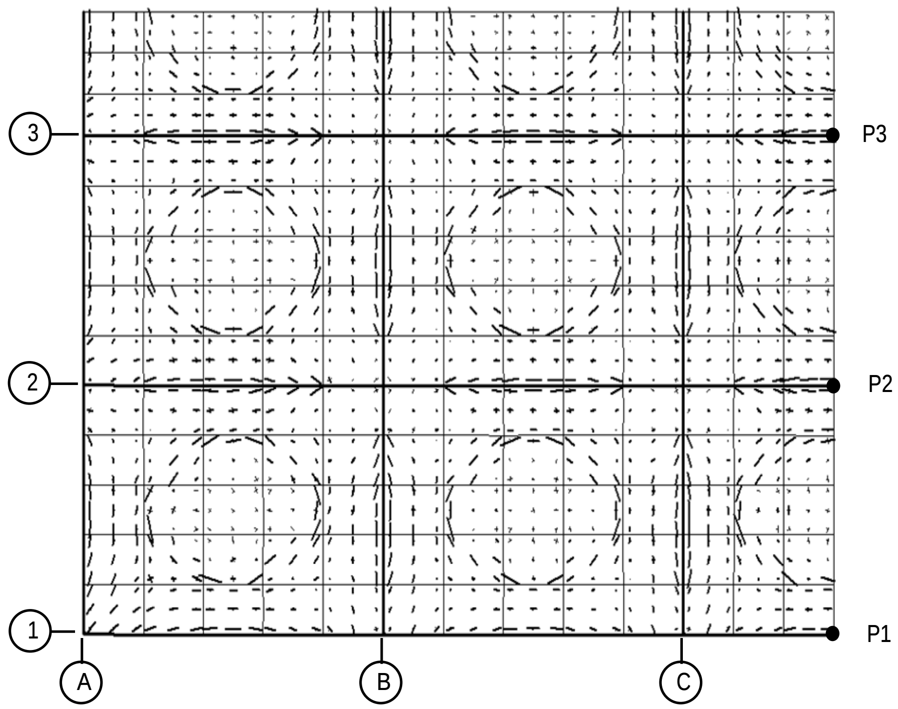

Fig. 8 - Distribution of principal membrane tractions at 150min (thick line=compression; thin line=tension).

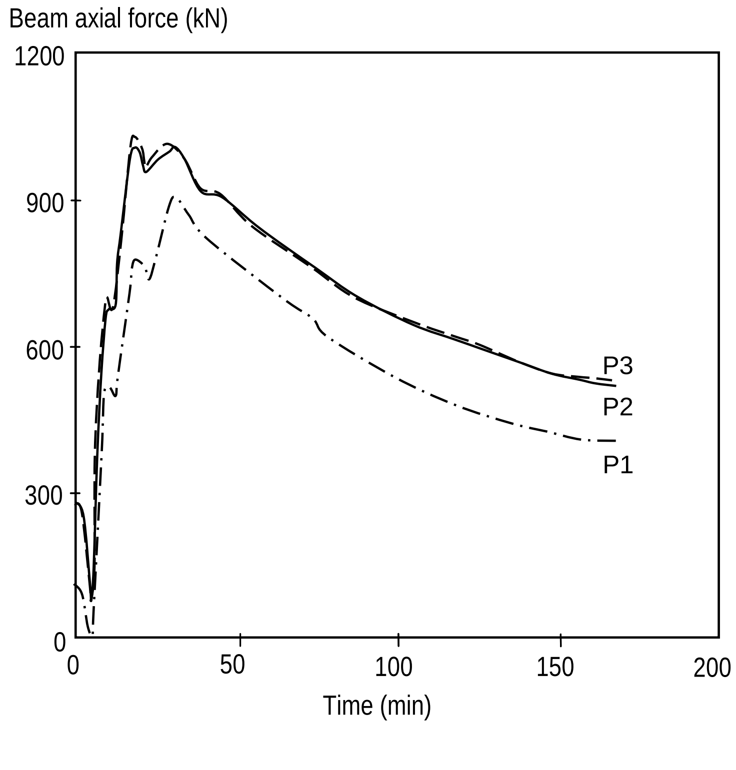

Fig. 9 - Tensile axial forces of beams at key positions.

4. STRUCTURAL BEHAVIOUR WITH FIRE COMPARTMENTS IN DIFFERENT LOCATIONS

In order to study the interactions between the cool and hot zones of the structure, a series of analyses has been carried out for different extents and positions of localised fire compartments. Three different locations are modelled, as indicated in Fig. 10. The temperature distributions for the structural members within the fire compartment are assumed to remain the same as above. The structure beyond the fire compartment is assumed to remain at 20°C.

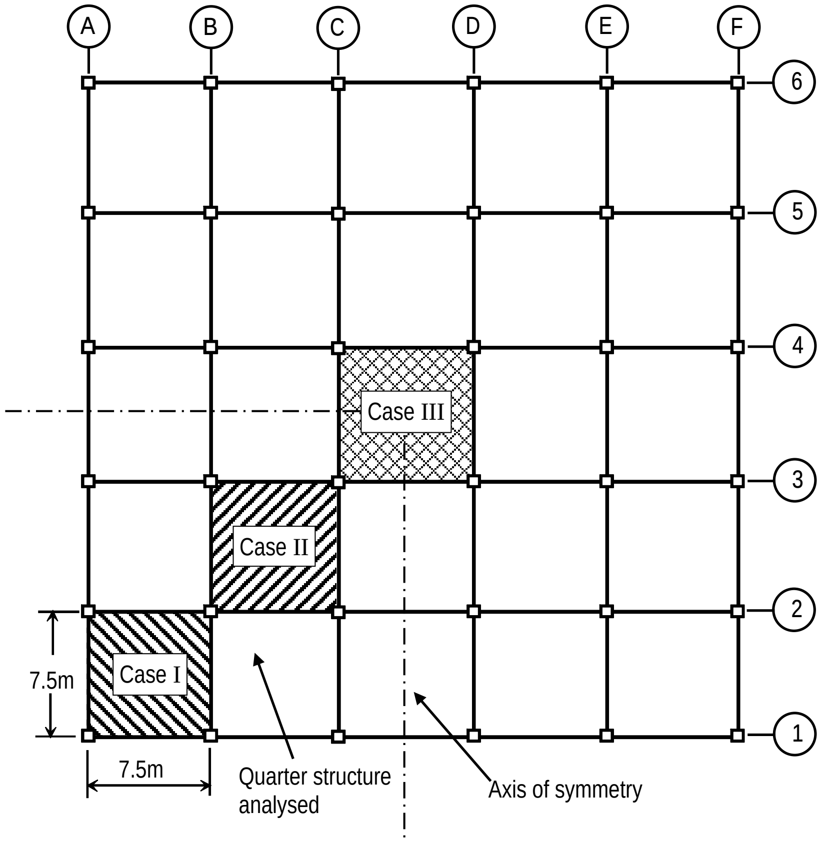

Fig. 10 - Concrete structure layout, with different fire compartment positions marked.

Fig. 11 shows the increase in deflection, at the centre of the fire compartment, with time for the three different cases. The time at which the deflection reaches 220mm is about 50min more for Case III (fire in an internal bay) than for Case I (fire in the corner bay). The structural behaviour within each compartment is clearly different, especially at high temperatures. This is because of the extent of restraint, from the adjacent cool structure and continuity of the floor slabs, which is greater for the internal bays than for the corner bays.

Figs. 12 to 14 show the distributions of the principal membrane tractions in the slab at 150min for Cases I, II and III. It is evident that no tensile membrane forces are formed for Cases II and III within the fire compartment, and that even for Case I only a very small portion of area within the fire compartment experiences tensile membrane forces.

It is clear from the above analyses that the presence or absence of adjacent cool slab areas has significant influence on the behaviour of the structure within the fire compartment. The restraint and continuity from this cool structure provide some benefits in increasing the fire resistance of the structure within the fire compartment. All analyses were eventually terminated due to buckling of heated columns. Once again, these confirm that prevention of column failure is most important in designing reinforced concrete structures for high fire resistance periods.

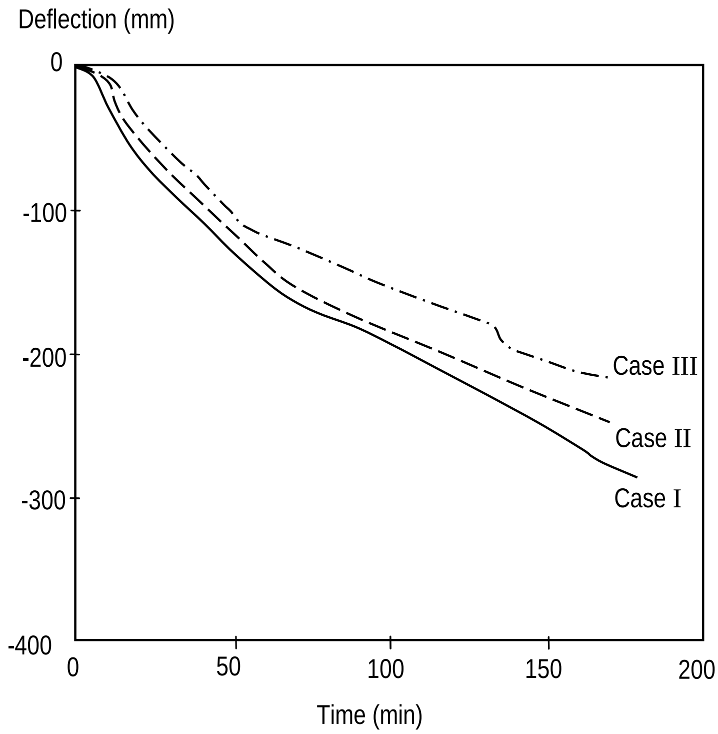

Fig. 11 - Comparison of predicted central deflections for different fire compartment positions.

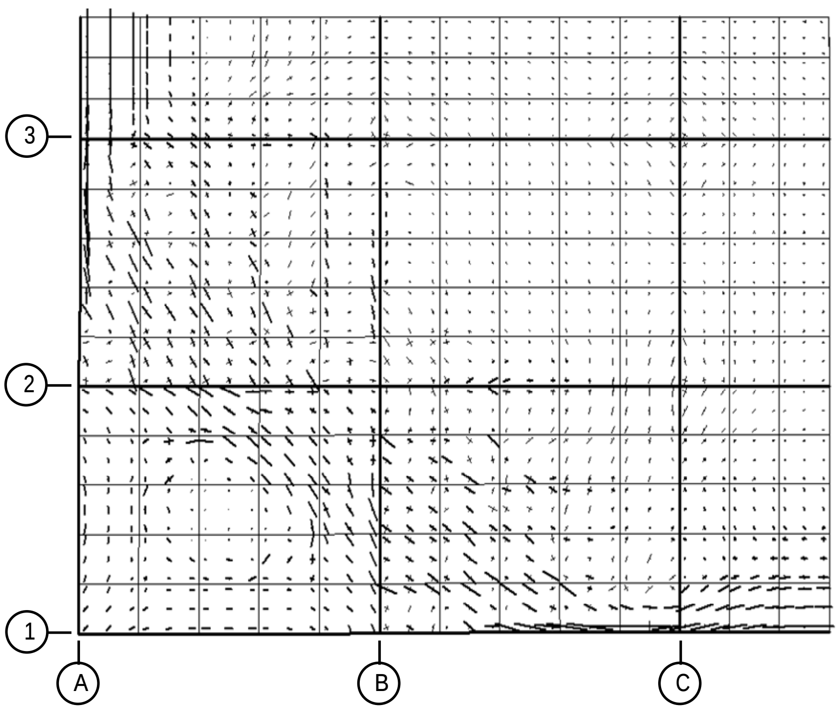

Fig. 12 - Case I: distribution of the principal membrane tractions at 150min (thick line=compression; thin line=tension).

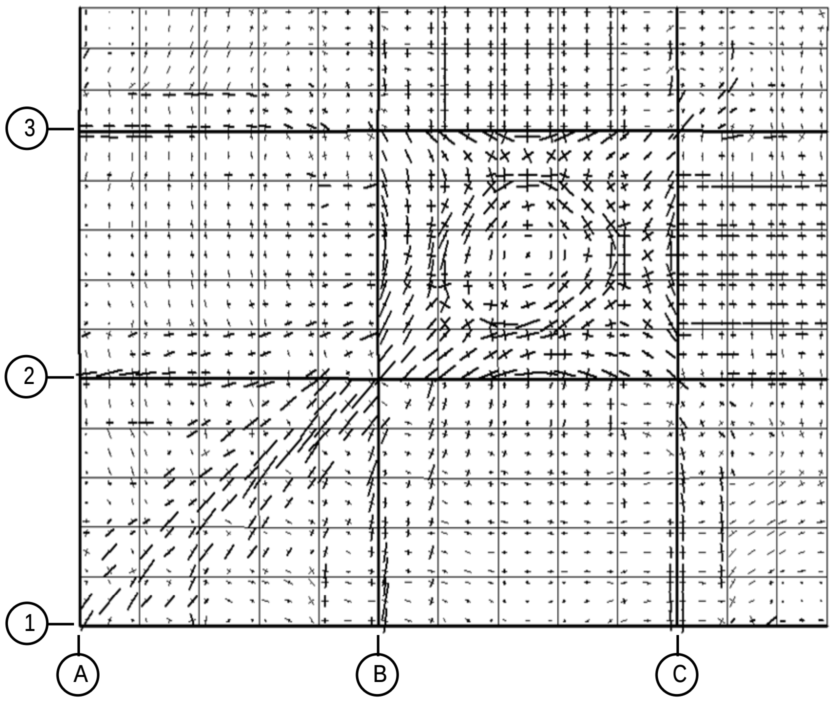

Fig. 13 - Case II: distribution of the principal membrane tractions at 150min (thick line=compression; thin line=tension).

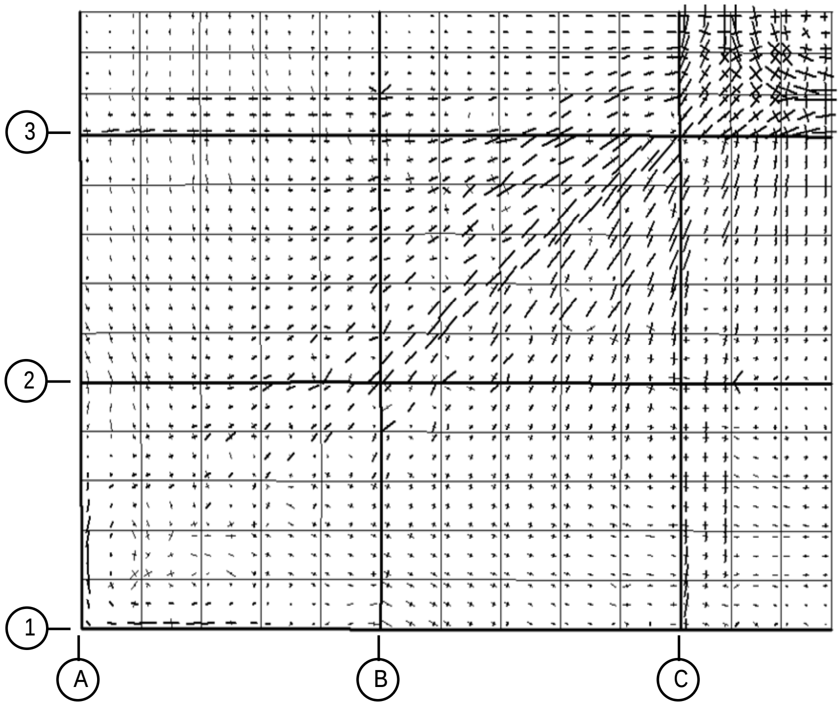

Fig. 14 - Case III: distribution of the principal membrane tractions at 150min (thick line=compression; thin line=tension).

5. CONCLUSIONS

In this paper a series of analyses has been carried out by using the computer program *Vulcan* on a generic reinforced concrete structure subject to the ISO834 standard fire. It is obvious that structural frame behaviour could be well represented through such FE analysis, which can provide information for rational design of reinforced concrete structures in fire. It is clear that adjacent cool structure provides restraint and continuity, increasing the fire resistance of the structure within the fire compartment. It is also clear from this study that relatively small areas of tensile membrane force were formed within the concrete slabs, and large areas of the slabs were subject to compressive membrane force during the fire. As a result the downstand concrete beams were subjected to enhanced tension during the fire, especially in the initial stages, and these tensions were mainly carried by their tensile reinforcement. It is therefore very important to keep the temperature of the reinforcement within certain limits.

The covers to reinforcement specified in current design codes seem reasonable provided that concrete spalling does not occur during the fire. Designers should therefore pay attention to measures to prevent concrete spalling, in order to enable structures to satisfy their fire resistance requirements. As for composite structures, the fire resistance of columns is vitally important for reinforced concrete buildings. In all cases analysed in this paper the eventual structural failures were due to buckling of the heated columns.

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