ANALYSIS OF COMPOSITE BUILDINGS UNDER FIRE CONDITIONS

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Abstract

In this paper, the performances of a generic three dimensional 45m x 45m composite floor subjected to ISO834 Fire and Natural Fire are investigated. The influences of reinforcing steel mesh and vertical support conditions on the tensile membrane action of floor slabs are investigated in details. Two robust 2-node connection element models developed by the authors are used to model the behaviour of end-plate and partial end-plate connections of composite structures under fire conditions. The impact of connections on the 3D behaviour of composite floor is considered. Based on the results obtained, some design recommendations are proposed to enhance the fire safety design of composite buildings.

Keywords: Steel Connection, Fire Resistance, Composite floor, Finite Element Modelling, Tensile Membrane Action.

1 INTRODUCTION

Observations from a series of full-scale Cardington fire tests have shown that steel framed composite structures can provide a significantly greater fire resistance than is suggested by standard fire tests on isolated members (Bailey et al.1999). This appeared to be due to an interaction between the heated members within the fire compartment, the concrete floor slabs and the connected steel frame structure. Experimental and analytical investigations involving full-scale fire tests indicated that tensile membrane action within the concrete floor slabs plays an important role in enhancing the fire resistance of composite buildings. However, the occurrence of tensile membrane action mainly relies on the conditions of vertical support maintained around the edges of the slab panel. The beams around the perimeter of the slab panels are needed to be protected, while the internal secondary beams can be left unprotected.

Up to now, research has focused on the influence of the tensile membrane actions on the fire resistance of composite floors, with the assumption that the fully vertical supports along the perimeter of the slab panel are provided by protected beams (Abu et al. 2013). However, there is as yet a lack of detail research into the influence of the vertical deflections of protected beams during fire on the tensile membrane actions of the slab panel. For the majority of previous researches on modelling composite floor subjected to fire, the beam-to-column and beam-to-beam connections were assumed to behave either as pinned or rigid for simplicity, and the vertical shear and axial tension failures of the connection were not taken into account (Huang et al. 2004).

This paper presents a comprehensive study conducted on a generic three dimensional 45m x 45m composite building, with realistic loading conditions and structural layout, under different fire conditions. A series of analyses has been carried out using different support conditions on floor slab panels and slab reinforcement details.

2 THEORETICAL BACKGROUND OF THE SOFTWARE VULCAN

In this study, the finite element software *Vulcan* is employed, which is capable to model the three dimensional performance of composite and steel-framed buildings under fire conditions. In this program, the steel-framed composite buildings are modelled as an assembly of finite beam-column, connection and slab elements. The beam-columns are modelled using 3-node line elements. The cross-section of each element is divided into a number of segments to allow the variation of the distributions of temperature, stress, and strain. The reinforced concrete slabs are represented using 9-node nonlinear layered elements, in which the membrane action of the floor slabs is considered.

The slab elements are divided into a number of plain concrete and reinforcing steel layers. The temperature and material properties for each layer can be specified independently.

Recently, two robust simplified connection models have been developed by the authors (Lin et al. 2013, 2014) for modelling the end-plate and partial end-plate connections between steel beams and columns in fire. The connections failure due to bending, axial tension, compression and vertical shear are all taken into account. The developed connection models have been validated against available experimental test data and incorporated into software *Vulcan*.

3 ANALYSIS OF 3D COMPOSITE FRAME UNDER DIFFERENT FIRE CONDITIONS

In this research, a series of numerical studies have been conducted on a generic three dimensional 45m x 45m composite building, under two typical fire conditions. As shown in Fig. 1, the floor consists of five 9m x 9m bays in each direction. For the steel beam members within the frame, the section of 533x210x92UB was used as the primary beam, while size of 356x127x39UB was adopted for the secondary beam with S355 steel. The column of size 305x305x97UC was applied with a height of 4.5m. The lightweight concrete composite floor had an overall depth of 130 mm, with PMF CF70 metal decking. It was assumed that the composite frame is designed for an office building, and two hours fire resistance is required. The total design load for this building was assumed to be $6.1\,kN/m^2$ at the fire limit state.

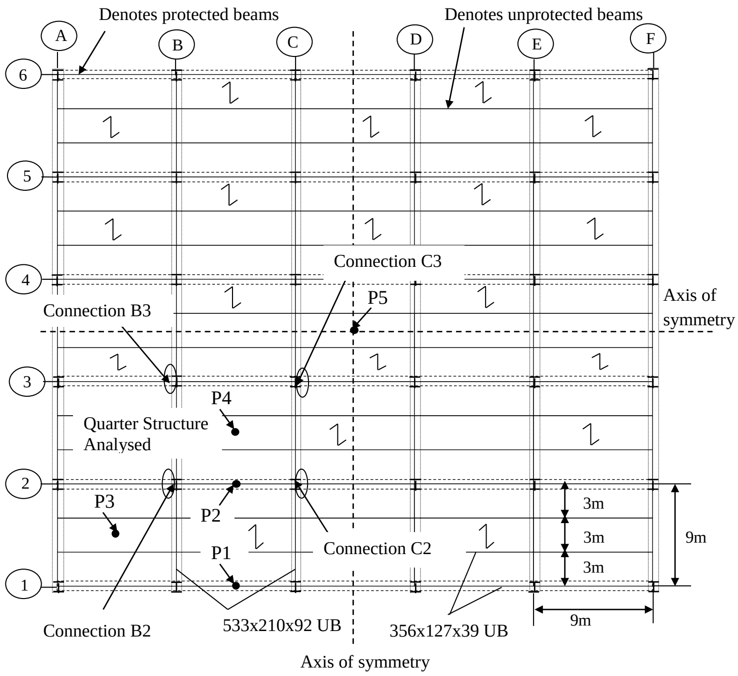


Fig. 1 Layout of 45m x 45m composite floor

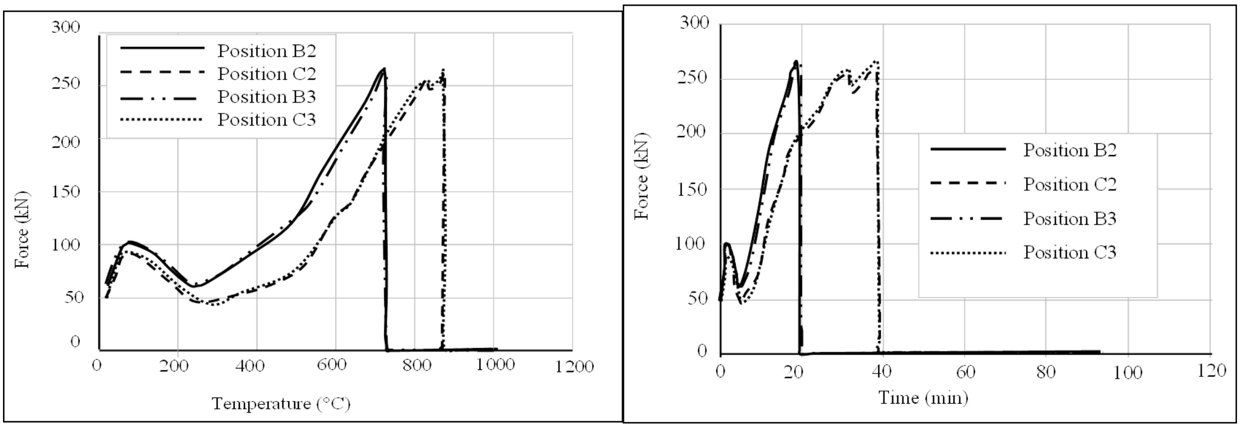
The flush end-plates connections (major-axis) were used to connect the primary beam to column flange. The partial end-plate connections (minor-axis) were used to connect the secondary beams to the columns' web, and secondary beams to the web of primary beams. For a flush end-plate connection, section of 573x250x25 was used with six M24 Grade 8.8 bolt rows. The vertical shear resistance of the connection at ambient temperature is $542\,kN$. For a partial end-plate connection, a size of 230x150x10 was applied with three M24 Grade 8.8 bolt rows. The vertical shear resistance of the connection at ambient temperature is $271\,kN$. Hence, the load ratio related to vertical shear for secondary beam is 0.3, while for the primary beam it is 0.15.

In this study, two different fire scenarios were adopted: ISO 834 Fire and Natural Fire. The Natural Fire was defined using a parametric temperature-time curve, calculated according to EN 1993-1-2 (2005).It was assumed that two hours fire resistance was required. All columns, primary beams and secondary beams along the column grid lines were fire protected, and all internal secondary beams were left unprotected. The maximum temperatures designed for unprotected secondary beams were 1047°C and 915°C respectively, under ISO 834 Fire and Natural Fire conditions. During the

cooling phases of the Natural Fire scenario, the designed minimum temperature was 114°C. For protected beams and columns, the maximum temperatures were less than 600°C and 550°C, respectively. The temperatures of connections were assumed to be 80% of the temperatures of the connected beams. The concrete slab was divided into fourteen layers. Each layer had a different uniform temperature distribution. It was assumed that the reinforcement was positioned just above the metal decking.
In this research, only one quarter of the frame was analysed, in order to save computing time by taking the advantage of symmetry. It was assumed that the whole ground floor of the building was under fire. A total of 15 cases were analysed using different steel meshes (A142, A252 and A393), under two different fire scenarios. In the following sections the reference temperatures for all figures are related to the temperatures of unprotected secondary beams within the fire compartment. Due to the limit space, only the cases under ISO 834 Fire condition will be presented in this paper.

3.1 The Impact of the Connections

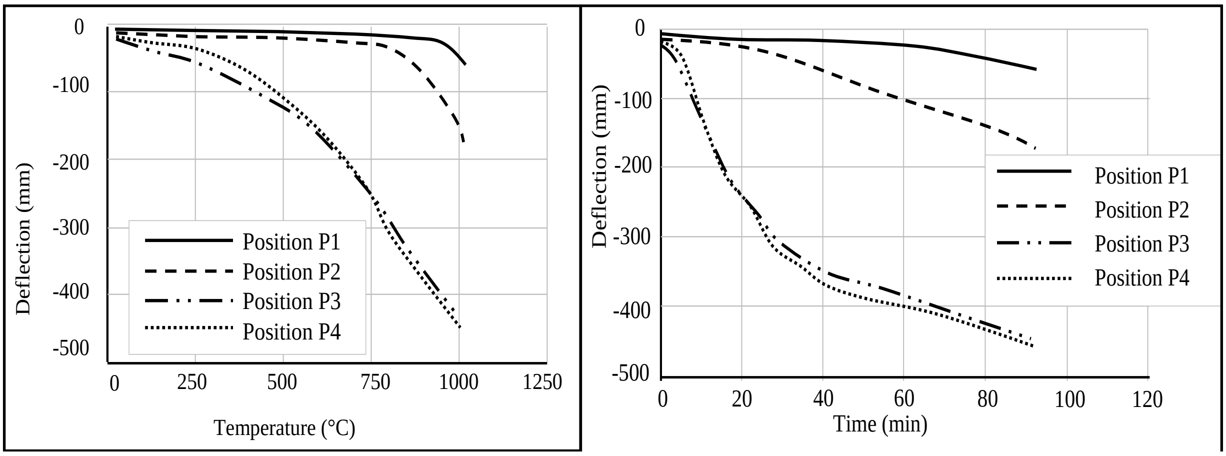
The first case was analysed using A142 steel mesh for floor slabs under ISO 834 Fire condition. Fig. 2 shows the predicted vertical shear forces of the partial end-plate connections, against temperature and time, for protected secondary beams at the positions B2, C2, B3 and C3 (see Fig. 1) under ISO Fire. These four connections connect the protected secondary beams to the columns. It can be clearly seen from the results that all the four connections failed due to vertical shear. As can be seen in Fig. 2, the vertical shear forces acting on the partial end-plate connections at B2, C2, B3 and C3 ranged from 50 to 60 kN at ambient temperature, which is less than 25% of the vertical shear resistance of the connections used. When temperatures of unprotected secondary beams were higher than 300 °C, these beams gradually lost their loading capacity, and the loads on the floor slabs were redistributed from the hot beams to the protected beams. When the unprotected beam temperature reached about 700 °C, the vertical shear forces acting on the partial end-plate connections at B2 and B3 increased to around 270 kN, exceeding the vertical shear resistance of the connections. Therefore, the connections positioned at B2 and B3 failed by vertical shear. The vertical supports of the protected secondary beams B2-C2 and B3-C3 were lost. When the temperature approached around 850 °C, the partial end-plate connections located at C2 and C3 failed due to the vertical shear. Then the vertical supports of the protected secondary beams C2-D2 and C3-D3 were also lost.



(a) (b)

Fig. 2 Predicted vertical shear forces of connections at different positions under ISO Fire: (a) connection shear force versus beam temperatures; (b) connection shear force versus time

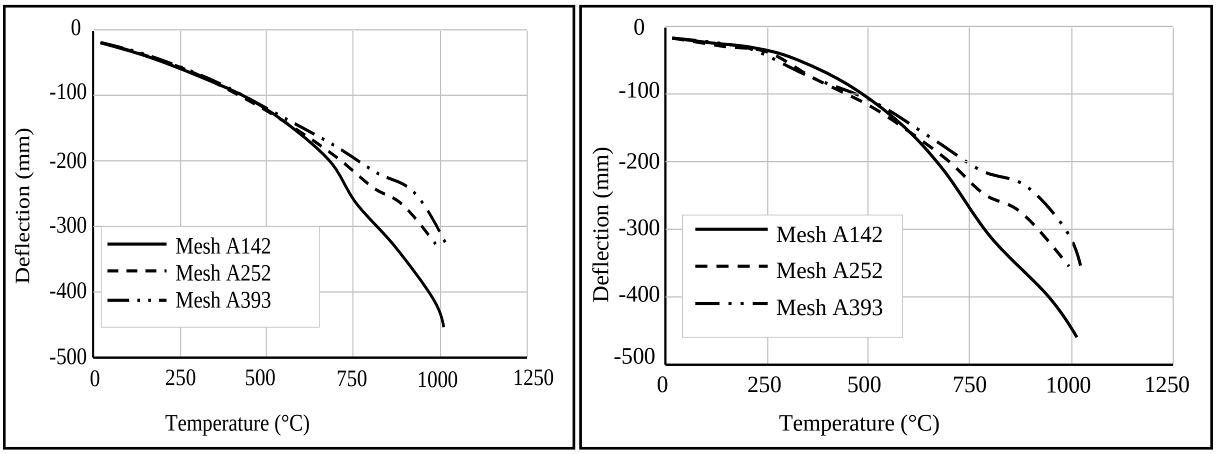
Fig. 3 shows the deflections versus temperatures and times at positions P1, P2, P3, and P4 (see Fig. 1), under ISO 834 Fire. As shown in Fig. 1, the positions of P1 and P2 are located at the mid-span of protected secondary beams, B1-C1 and B2-C2, while P3 and P4 are positioned at the centres of the compartments. It can be observed that the deflections at the mid-span of the protected beams increased significantly, when the temperature of unprotected beams was beyond 800 °C. Therefore, the vertical support for the floor slabs panels, provided by the protected beams, was significantly reduced. Because of the significant deflections of protected secondary beams, the floor slab panels deformed less double curvature. Hence, the tensile membrane actions within the floor slabs were then reduced considerably.



(a) (b)

Fig. 3 Predicted deflections at different positions under ISO Fire: (a) deflection versus beam temperatures; (b) deflection versus time

It is clear from the above analysis that the behaviour of connections positioned at the perimeter of slab panels has a significant influence on the formation of tensile membrane action. In this research, the load factor of the vertical shear of connections between protected secondary beams and columns is 0.3. However, in this case the connections still failed, due to loads transferred from unprotected beams to the protected beams as the fire developed. The failure of connections connected to the protected beams reduced the vertical support to the slab panels. Hence, the positive influences of tensile membrane action within the floor slab panel were significant reduced. Therefore, for the structural fire engineering design, if the designers want to leverage the benefits of tensile membrane action on the performance of composite floors in fire, the vertical shear capacity of the connections between protected secondary beams and columns needs to be significant increased compared to normal design.



(a) (b)

Fig. 4 Predicted deflections under ISO Fire using different steel meshes: (a) at position P3; (b) at position P4

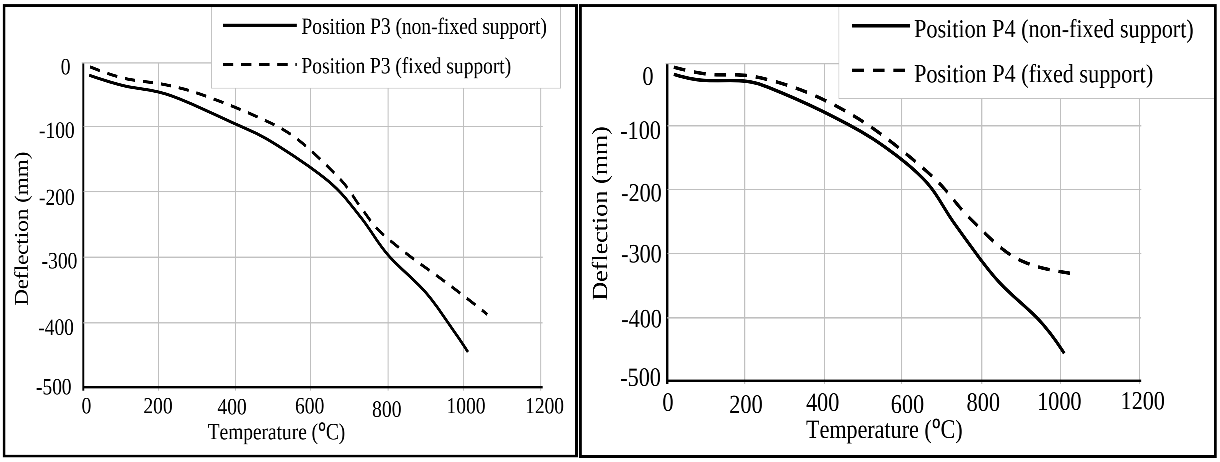
3.2 Influence of Reinforcing Steel Mesh

Three different steel meshes (A142, A252 and A393) were used in this research, to demonstrate the effect of steel reinforcement on the structural behaviour of floor slabs under fire conditions. The predicted deflections at positions P3 and P4 under ISO834 Fire condition are illustrated in Fig. 4. It can be observed that the impact of different slab reinforcement is negligible, up to 500 °C. But beyond 500 °C, the differences between the three steel meshes become more obvious. When the steel beams lose strength and stiffness at high temperatures, the concrete slab plays a more important role in supporting the loads. The ultimate load-carrying capacity of concrete slabs largely depends on the reinforcement area and strength. Therefore, at high temperatures, the impact of steel reinforcement becomes more significant.

3.3 Influence of Vertical Support of Protected Beams

In order to quantitatively assess the influence of the vertical support provided by the protected beams along the edges of the slab panel on the membrane actions of the floor slabs, the case with A142 mesh under ISO fire condition was reanalysed with all protected beams fixed vertically. This means that all protected beams along the column grid lines have no vertical deformation, and the slab panels were fully vertically supported along the edges.

Fig. 5 demonstrates the comparisons of the displacements at positions P3 and P4, with fixed and non-fixed vertical support on protected steel beams. For the non-fixed case the protected steel beams were free to deflect vertically as the normal situation. From the comparison results, it can be noticed that the discrepancy between these two cases increases after the temperature of unprotected beams reaches 700^{0}C. Beyond this temperature, the unprotected secondary beam loses strength progressively, leading to the loads above the slab panel being redistributed to the protected edge beams along the slab panels. For the normal case (with non-fixed support) the protected secondary beams were vertically deflected significantly due to the increased loads, especially when the temperature of unprotected beams was higher. Therefore, the slab panels of the fire compartment deformed less double curvature and the tensile membrane action of the slab panel was considerably reduced. In comparison for the case with fixed support on the protected beams, the slab panel was fully vertically supported along the edges of the slab panel. Therefore, the slab panel was forced to deform double curvature hence the tensile membrane action within the slab panel was fully maximized. In this situation, the load capacity of the slab panel considered was significantly enhanced due to the tensile membrane action.



(a) (b)

Fig. 5 Comparison of predicted deflections subjected to ISO Fire with non-fixed and fixed vertical support on protected steel beams (A142 mesh): (a) at position P3; (b) at position P4.

The investigation reported above demonstrated that the influence of the vertical support condition on the tensile membrane action of floor slabs is significant. It is important for fire structural engineering designers to account for this issue, when they attempt to utilise the tensile membrane action of floor slabs for their design.

The numerical studies described above show that the provision of vertical support along the slab panel has a significant impact in the formation of tensile membrane action within the floor slabs under fire conditions. At a higher temperature, the loads within the floor slabs of fire compartment (initially supported by unprotected secondary beams in the fire compartment) will be redistributed into protected beams along the column grid lines. This large load will cause the protected secondary beams to deform significantly. Also, the high load will result in vertical shear failure of the connections which connect protected secondary beams to the columns. All of this will significantly undermine the vertical support conditions on floor slab panels within the fire compartment. In return, the benefit of tensile membrane action for enhancing the load carry capacity of the floor slab panels, as initial assumed in the performance-based design, could be considerable reduced.

Therefore, the load-carrying capacities of connections between protected beams and columns need to be adequately designed to resist larger vertical shear forces compared to normal design. The primary requirements for effective use of the tensile membrane action of the floor slabs is to make sure that the strong vertical supports along the edges of floor slab panels are maintained during the required fire resistance period. Floor slab panels are forced to deform, as the double curvature shape. If the protected edge beams undergo excessive deformation, the floor slab panels may convert into a single-curvature deformation, which will reduce the benefit of tensile membrane action significantly. Therefore, the protected beams along the perimeter of the slab panels should be designed carefully, to provide sufficient vertical support during fire. Larger cross-section sizes of protected secondary beams are needed compared to normal design. A higher reinforcement ratio for floor slabs may also be used to enhance the positive influence of the tensile membrane action to improve the fire resistance of the composite floor system.

4 CONCLUSIONS

The research conducted here systematically investigated the impact of the connections for protected beams on the tensile membrane actions of supported floor slabs, in which the failure of the connections due to axial tension, vertical shear, or bending was considered. The influence of the vertical deflections of protected beams on the tensile membrane action of the floor slabs has been analysed in detail. The effects of different reinforcement details of the floor slabs, on the performance of a composite floor under different fire conditions, were also evaluated.Based on the results of this investigation, the following conclusions can be drawn:

- At higher temperatures, the loads within the floor slabs of fire compartment, initially supported by unprotected secondary beams, are redistributed into protected beams along the column grid lines. This large load causes the protected secondary beams to deform significantly.
- The high loads result in vertical shear failure of the connections, which connect protected secondary beams to the columns. All of this will significantly undermine the vertical support conditions on floor slab panels within the fire compartment.
- The designers should design adequate strength and stiffness into protected secondary beams and connections. The primary requirements for tensile membrane action of the floor slabs to be effective used, is to make sure that the strong vertical supports along the edges of floor slab panes are maintained during the required fire resistance period.

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