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<td><strong>Author(s)</strong></td>
<td>Tan, K.-H.; Nguyen, T.-T.</td>
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Abstract: This paper presents novel experimental results and observations from three one-quarter scale tests on two-way concrete slabs supported by protected steel edge beams under fire conditions. The sizes of the protected secondary edge beams were varied to study the effect of beam stiffness on the fire behaviour of the assemblies. Test results showed that as the stiffness of the protected secondary edge beams increased, the slab central deflection decreased and failure of the slab occurred later. However, composite action between the edge beams and the concrete slab plays a key role in mobilising this beneficial effect. Once the composite slab-beam action is weakened by cracks in the slab over the main or secondary edge beams, the benefit associated with a greater stiffness of the edge beams is lost. Tensile membrane action was mobilised at a deflection equal to 0.9 to 1.0 of the slab thickness irrespective of the bending stiffness of the edge beams. The commencement of tensile membrane stage was marked by one of three indicators: (a) concrete cracks which formed a peripheral compressive ring in the slab; (b) horizontal in-plane displacements along the slab edges; and (c) horizontal and vertical displacements of four corner protected steel columns. The test results were used to validate a finite element model developed using Abaqus/Explicit. Good correlation between the predicted and experimental results was obtained.

Response to Reviewers: Response to Reviewer comment No. 1: The authors thank the Reviewer for valuable discussions.

Response to Reviewer comment No. 2: The authors thank the Reviewer for valuable comments and discussions. The authors would like to address the comments as follows.

Q1: "The description of the displacement history of the columns is unclear. The initial expansion results in an upwards displacement as expected, but it is stated that this is reversed due to the pulling effect of TMA. However, this would surely result in a predominantly horizontal displacement, and there is no mention of the softening of the column due to further heating. This needs some clarification." The authors agree that in the first stage, the column gradually moved upwards to its original position due to thermal expansion only. At this stage, there was not due to pulling-in of the edge beam ends since the slab deflection was still small. The following paragraph was revised to clarify it.
In the first stage, when heating started, the column gradually moved upwards due to thermal expansion, and then returned to its original position due to pulling-in of the edge beam ends. For instance, the column in P215-M1099 moved back to its original position after 23.7 min.

In the second stage, the columns moved downwards again as shown in Fig. 9(b). This was attributed by two reasons. One was caused by pulling-in of the edge beam ends; the other was caused by softening of the column due to further heating.

The discussion concerning the effect of changing the beam stiffness and the resulting deflections would benefit from a little more emphasis, particularly as there is something of an anomaly when considering the effect of increased beam stiffness on deflections. This is put down to the loss of composite action in the case of specimen P368, but it is unclear why this should have lost composite action earlier than the other beams, and what evidence there was to support this proposition. The conclusions are, however, do provide a fair reflection of the observations.

All three specimens experienced a similar crack pattern at the end of the tests with severe cracks in the vicinity of the edge beams as shown in Fig. 11. However, the time at which these cracks occurred was unpredictable and different for different specimens. Therefore, unfortunately it was inexplicable why in the case of specimen P368-M1099, the composite action has lost earlier than the other beams. It may be caused by some errors during casting of P368-M1099.

The severe cracks which weakened the composite action in P368-M1099 test are shown in Fig. 11. This figure provides solid evidences to support the proposition. Once the composite action had been weakened, the benefit associated with a greater stiffness of the edge beams was lost.

The following paragraph was revised to clarify it. Original (Section 3.2 – 2nd paragraph, last sentence): “...Consequently, composite action between the main beam and the concrete slab was weakened. Therefore, the beneficial effect of increasing the stiffness of PSB was lost.”

Revision (Section 3.2 – 2nd paragraph, last sentence): “...Consequently, composite action between the main beam and the concrete slab was weakened, and the beneficial effect of increasing the stiffness of PSB was lost. However, the time at which the cracks in the vicinity of the edge beams occurred was unpredictable and different for different specimens. Therefore, it was inexplicable why in the case of specimen P368-M1099 the composite action had lost earlier than the other beams.”
Dear Editors,

We would like to submit the revised enclosed manuscript entitled “Experimental and Numerical Evaluation of Composite Floor Systems under Fire Conditions”, which we wish to be considered for publication in “Journal of Constructional Steel Research”.

In this work, we tested three one-quarter scale composite slab-beam systems under fire conditions and simulated the results using Abaqus. Since the results were extensive, it is not easy to combine the figures. Therefore as many as 15 figures and 4 tables were included. The estimated word number is about 7,000.

The authors deeply appreciate the reviewers’ comments on our manuscript. All the comments have been carefully studied. The corresponding responses and amendments have been included in the attached file of replies to reviewers’ comments as well as in the revised manuscript.

We deeply appreciate your consideration of our revised manuscript. If you have any queries, please contact me at the address below.

Thank you and best regards.

Yours sincerely,

Tuan-Trung, NGUYEN

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Highlights

- Tested 3 composite floor assemblies in fire to study stiffness of secondary beams.
- Abaqus/Explicit was used to simulate results with good correlation.
- Stiffness of the secondary edge beams increased, the slab deflection decreased.
- Composite action between the edge beams and the concrete slab plays a key role.
- The commencement of tensile membrane stage was marked by one of three indicators.
Reviewer #1: This paper presents experimental and numerical work aimed at understanding the behaviour of composite floor systems in fire. Three tests have been carried out as part of this study. These are well formulated and well reported, and consider variations in the setup relevant to real slab systems. A numerical model was developed in Abaqus/Explicit and used to predict the response of the slabs varying the restraints provided along the edges. Overall the paper is well written and organised, and can be accepted in the current form.

The authors thank the Reviewer for valuable comments and discussions.
Reply to Reviewer #2

The authors thank the Reviewer for valuable comments and discussions. The authors would like to address the comments as follows.

1. Overall this is a well written paper dealing with a topic of considerable interest. The description of the displacement history of the columns is unclear. The initial expansion results in an upwards displacement as expected, but it is stated that this is reversed due to the pulling effect of TMA. However, this would surely result in a predominantly horizontal displacement, and there is no mention of the softening of the column due to further heating. This needs some clarification.

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Original (Section 3.3 – 4th paragraph): “…In the first stage, when heating started, the column gradually moved upwards due to thermal expansion, and then returned to its original position due to pulling-in of the edge beam ends. For instance, the column in P215-M1099 moved back to its original position after 23.7min...”

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was caused by softening of the column due to further heating. The following paragraph was revised.

*Original (Section 3.3 – 5th paragraph):* “In the second stage, when TMA was mobilised, the columns were pulled in due to tensile forces in the edge beams. Therefore, the columns moved downwards again, leading to an upward trend in the displacement-time curves (Fig. 9(b))…”

*Revision (Section 3.3 – 5th paragraph):* “In the second stage, when TMA was mobilised, the columns were pulled in due to tensile forces in the edge beams. The softening of the columns also occurred due to heating. Therefore, the columns moved downwards again, leading to an upward trend in the displacement-time curves (Fig. 9(b))…”

2. *The discussion concerning the effect of changing the beam stiffness and the resulting deflections would benefit from a little more emphasis, particularly as there is something of an anomaly when considering the effect of increased beam stiffness on deflections. This is put down to the loss of composite action in the case of specimen P368, but it is unclear why this should have lost composite action earlier than the other beams, and what evidence there was to support this proposition. The conclusions are, however, do provide a fair reflection of the observations.*

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The severe cracks which weakened the composite action in P368-M1099 test are shown in *Fig. R1*. This figure provides solid evidences to support the proposition. Once the
composite action had been weakened, the benefit associated with a greater stiffness of the edge beams was lost.

**Fig. R1** Severe cracks above edge beams of P368-M1099

The following paragraph was revised to clarify it.

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Experimental and Numerical Evaluation of Composite Floor Systems under Fire Conditions

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Abstract

This paper presents novel experimental results and observations from three one-quarter scale tests on two-way concrete slabs supported by protected steel edge beams under fire conditions. The sizes of the protected secondary edge beams were varied to study the effect of beam stiffness on the fire behaviour of the assemblies. Test results showed that as the stiffness of the protected secondary edge beams increased, the slab central deflection decreased and failure of the slab occurred later. However, composite action between the edge beams and the concrete slab plays a key role in mobilising this beneficial effect. Once the composite slab-beam action is weakened by cracks in the slab over the main or secondary edge beams, the benefit associated with a greater stiffness of the edge beams is lost. Tensile membrane action was mobilised at a deflection equal to 0.9 to 1.0 of the slab thickness irrespective of the bending stiffness of the edge beams. The commencement of tensile membrane stage was marked by one of three indicators: (a) concrete cracks which formed a peripheral compressive ring in the slab; (b) horizontal in-plane displacements along the slab edges; and (c) horizontal and vertical displacements of four corner protected steel columns. The test results were used to validate a finite element model developed using Abaqus/Explicit. Good correlation between the predicted and experimental results was obtained.

Keywords: Tensile membrane action; Slab-beam systems; Restraint; Composite slabs; Fire.

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1 Introduction

It has been experimentally observed that the ultimate load of composite slabs in steel-framed buildings under fire conditions is significantly greater compared to the load-carrying capacity predicted by the conventional yield-line theory. The increase in the ultimate load is due to the contribution of tensile membrane action (TMA) which develops in the composite slabs at large deflections. This beneficial effect allows secondary interior steel beams to be unprotected, providing savings of fire protection materials for steel-framed composite buildings.

The Cardington fire tests performed by the British Research Establishment and British Steel between 1995 and 2003 [1] triggered the first research wave on tensile membrane behaviour of composite slabs in fire. Numerous research works have been conducted and only a few key references are mentioned here [2-7]. Although these works have been very valuable in providing a great understanding of membrane behaviour of composite floors in fire, most of these studies focused on isolated concrete slabs, rather than on composite slab-beam systems.

Recently, there has been a renewed wave of interest in the membrane behaviour of integrated floor assemblies in fire, which consist of protected edge beams and supporting columns, unprotected interior beams, and a composite slab. Most of these tests demonstrated very good fire performance of floor assemblies which relied on TMA in the slab mobilised at large deformation stage. Zhao et al. [8] conducted a full scale test on a single composite floor slab panel, 6.66m x 8.74m, representative of a corner compartment. The slab panel, which consisted of protected edge beams with two-hour fire rating and two unprotected secondary beams, was tested under exposure to a two-hour ISO 834 standard fire. The fire resistance was over two hours. The floor system did not collapse although the failure was due to poor welding between two steel reinforcing meshes.
Zhang et al. [9] tested four 5.23m x 3.72m composite slabs subjected to ISO 834 standard fire. Two tests had one unprotected interior beam and another two without interior beams. The authors observed that no structural collapse was found and membrane action was mobilised to carry the applied load. They concluded that interior secondary beams were not needed to support the slab under fire conditions and could be left unprotected. However, in all their tests, the edge beams were outside the furnace and were not heated directly.

Wellman et al. [10] conducted three tests to study the behaviour of composite floor assemblies subjected to standard and non-standard fire conditions with uncontrolled or controlled cooling phase. The specimens were designed with two different connection configurations and two different fire protection scenarios (interior beams with or without fire protection). The authors observed that both protected connection types (shear tab and double angle) did not fail during the heating or cooling stage. They concluded that the interior secondary beams could be left unprotected provided that a better load-transfer mechanism from the secondary to primary beams is available, for example, by increasing the slab thickness.

Some studies on membrane behaviour of composite slabs incorporating cellular steel beams have also been conducted [11, 12]. Nadjai et al. [11] conducted a large-scale natural compartment fire test on a 9.6m x 15.6m composite floor slab supported by long-span cellular beams. The tested slab was supported on a steel frame spanning 9m by 15m between four columns. All the columns and the edge beams were protected, while the interior cellular beams were unprotected. It is found that the reinforcement in the central region of the slab was under tensile force and a concrete compressive ring was formed around the perimeter of the slab. They concluded that the interior secondary beams can be left unprotected due to mobilisation of tensile membrane action.
Although previous studies offer valuable insight into the fire behaviour of composite slab-beam systems, there has not been any reported experimental study on the effect of stiffness of protected edge beams. In literature, there was only a numerical study on this issue. Lim [13] conducted a numerical study on the fire behaviour of the slab-beam systems with different sizes of supporting edge beams. He concluded that as the beam size decreased, failure of the slab occurred earlier with greater deflections. However, the positive effect of the beam size on the slab behaviour was only confirmed by numerical studies in which concrete cracking and crushing could not be accurately modelled.

To bridge this gap, a series of tests have been conducted at Nanyang Technological University, Singapore in 2012. This paper first presents the experimental behaviour of three one-quarter scale composite beam-slab floor systems in fire. The test results were then used to validate a numerical model developed in Abaqus/Explicit. The main parameter was the bending stiffness of protected secondary edge beams.

2 Test Setup

2.1 Test specimens

Numerical studies [14] showed that in terms of the four geometric properties of the protected edge beams (steel grade, torsional rigidity \(Gl\), bending stiffness about major axis \(EI_y\), bending stiffness about minor axis \(EI_z\)), only the bending stiffness about the major axis \(EI_y\) has significant effect on membrane behaviour of floor assemblies. Therefore, the bending stiffness about the major axis \(EI_y\) was chosen as the main parameter in this experiment.

The experiment consisted of three one-quarter scale composite beam-slab systems tested at elevated temperature, which were denoted as P215-M1099, P368-M1099 and P486-M1099. In this nomenclature, for example, P215-M1099 indicates a specimen which has 215 cm\(^4\) as the second moment of area about the major axis of protected secondary edge beam \((I_{PSB})\), and 1099 cm\(^4\) as that of main edge beam \((I_{MB})\). P215-M1099 was chosen as the
control specimen. \( I_{yPSB} \) of P368-M1099 and P486-M1099 was respectively increased to 1.71 and 2.26 times of \( I_{yPSB} \) of P215-M1099. The protected main and unprotected interior secondary beams for these specimens were kept the same. The effect of two unprotected interior beams on tensile membrane behaviour of beam-slab systems has been investigated separately [15].

This experiment applied the fire protection strategy for members recommended in the SCI Publication P288 [16]. All the edge beams and columns were protected to a prescriptive fire-protection rating of 60min. No fire-proofing material was applied to the interior beams and the slabs.

Fig. 1 shows a typical specimen with the slab 2.25m long by 2.25m wide and an outstand of 0.45m around the four edges. Along each edge were five M24 bolts with half of these bolt lengths cast into the slabs, while the other half were attached to an in-plane restraint system shown in Fig. 2. The locations of these bolts were fixed by using 8mm thick steel plates along the four slab edges. The purpose of these bolts was to simulate accurately the boundary conditions of interior slab panels. The interior slab panels should be rotationally restrained and could only have horizontal straight movement along the four edges as explained in Section 2.2.

The slab thickness was 57mm, 58mm, and 55mm for P215-M1099, P368-M1099, and P486-M1099, respectively. Shrinkage reinforcing mesh with a grid size of 80mm x 80mm and a diameter of 3mm (giving a reinforcement ratio of 0.16%) was placed within the slab, 25mm from the top. The mesh was continuous across the whole slab with a yield strength of 689MPa and an ultimate strength of 806MPa. Ultimate strain of the mesh was 14.8%, and the elastic modulus was 203.4GPa. The specimens were cast using ready-mixed concrete with the aggregate size ranging from 5 to 10mm. Six cylinders (150mm in diameter and 300mm long)
were tested at 28 days after casting giving a mean compressive strength $f_{cm}$ of 31.3MPa, 32.9MPa and 28.9MPa for P215-M1099, P368-M1099 and P486-M1099, respectively.

All the selected steel beams were Class 1 sections according to EN 1993-1-1 [17]. Fabricated section was used for secondary beams when necessary. The beams were designed for full-shear composite action using 40mm long, 13mm diameter headed shear studs with a spacing of 80mm to prevent failure due to shear. This was successful since there was no observed failure of shear studs. Flexible end plate joints were used for both beam-to-beam and beam-to-column connections. Both the beam-to-beam and beam-to-column connections were fire-protected. The protected columns were selected to be very stiff (UC 152x152x30) to avoid any instability failure.

Tensile coupon tests, two from the web and two from the flanges for each type of I-section used, were conducted at ambient temperature. Table 1 summarises the average results from the tensile tests and geometrical properties of the protected main beams (MB), protected secondary beams (PSB), and unprotected secondary beams (USB). All specimens had two interior USB consisting of fabricated I-sections of 80x80x17.3. The numerals of 80x80x17.3 refer to a steel section of 80mm deep, 80mm wide, with a mass of 17.3kg/m.

2.2 Test rig

The test setup arrangement is shown in Fig. 2. An electric furnace of dimensions 3m long by 3m wide by 0.75m high was setup. Its dimensions were dictated by the space constraint of the fire laboratory. Although the furnace could not simulate the ISO 834 fire curve due to limit of power, from the trial tests its air temperature could attain 1000°C within 50min, i.e. at a heating rate of about 20°C/min, which was within the practical heating rate for steel sections as prescribed in BS 5950-8 [18] which specifies a value from 5 to 20°C/min.

The concrete slab was placed on top of the furnace and then heated from the bottom. The supporting beam systems were totally enclosed within the furnace and heated from three
sides. Four protected I-section columns were rigidly connected to four fire-protected supporting circular columns by rigid connections. These circular columns were located outside the furnace, and connected to the strong floor by hinged connections which allowed the specimen to sway horizontally without any restraint (Fig. 2).

Uniformly distributed load was simulated using a 12 point loading system which consisted of three rectangular hollow section (RHS) beams and four triangular steel plates. Each steel plate had three points in contact with the slab through three M24 bolts. When the slab deformed, verticality of the loading system was ensured by the ball-and-socket joints placed in between the steel plates and RHS beams.

The specimens were set up with two restraint beam systems (Fig. 2). The first system was the rotational restraint system which consisted of four 160x100x6 RHS beams placed on top of the slab specimen and fixed to the reaction frame via two triangular stiffeners. In this experiment, it was assumed that reinforcement continuity over the supporting edge beams resulted from shrinkage mesh and the 0.45m slab outstand provided very little rotational restraint, since there was only one layer of shrinkage mesh placed inside the concrete slab.

The second system was the ‘so-called’ in-plane restraint system, which also consisted of four 160x100x6 RHS beams. This system was directly fixed to the four slab edges via five M24 bolts along each edge at a spacing of 750mm (Fig. 1). The in-plane restraint system was connected to the rotational restraint system by a different line of bolts. These two systems aimed to simulate accurately the boundary conditions of interior slab panels. The in-plane restraint system allowed the slab edges to translate inwards or outwards in straight edges, while the rotational restraint system applied flexural restraint on the slab edges. It is worth noting that there was a 20 mm gap between the in-plane restraint system and the furnace walls to avoid transferring the load to the latter. The gap was filled by insulation materials to avoid heat loss.
It is of great interest to find out if the simulated boundary conditions of an interior slab panel actually work. Considering a composite steel-framed building, there are two possible fire scenarios. *The common scenario* is one where a fire spreads throughout the whole soffit of the floor (Fig. 3(a)). In this situation, the heated slab would initially move outwards due to thermal expansion. It then moves inwards resulting from tensile membrane action mobilised at large deflections. However, the common boundary between two interior slab panels must translate in straight edges to ensure displacement compatibility. Therefore, in this case the slab can only move outwards and then inwards in straight edges. *In the second fire scenario*, if a fire is localised within one compartment (Fig. 3(b)), the ‘cool’ zones around the heated zone would restrain horizontal movement of interior slab panels due to thermal expansion, causing inplane restrained forces. Stiffness of inplane restraint from the ‘cool’ zone depends on many factors, such as the number and dimensions of unheated slab panels, the positions and dimensions of shear walls, etc. Thus, it is not possible to investigate these factors experimentally in the current test set up.

Therefore in this research, the authors aimed to model accurately an interior slab panel in the first fire scenario as shown in Fig. 3(a). Due to the two restraint beam systems, all specimens can be considered as interior slab panels which are restrained rotationally. Besides, the slab edges will translate in straight lines. Test results shown in Section 3.2 indicate that this objective was indeed achieved.

2.3 Instrumentation and Test load

K-type thermocouples and linear variable differential transducers (LVDT) were used to measure temperatures and displacements of the beams and the slab. A similar set of instrumentation was used for all specimens. Temperature of the slab was measured at Sections 1, 2 and 3 (Fig. 4(a)). At each section, temperature was monitored at the top and bottom surfaces, and at the reinforcing mesh level. Temperatures of the beams were
measured at Sections A to F at the top and bottom flanges, and at the middle of the beam web. The furnace air temperature was also monitored.

A total of 20 LVDTs were used to measure displacements of the floor assemblies (Fig. 4(b)) in which L1 to L11 were used to record vertical deflections of the slab and the beams. L12 to L14 were used to measure horizontal and vertical displacements of a column, while L15 to L20 for horizontal displacements along the slab edges.

The slabs were initially loaded up to 15.8kN/m² and then heated up to failure. Load was kept constant during the heating stage. After failure had been identified, the load was removed and natural cooling began. The test load of 15.8kN/m² was equal to 0.35 times the conventional yield-line load at ambient temperature, which was 45.1kN/m² based on the slab configuration with two interior steel beams. The moment capacity of supporting beams does not affect the yield-line load since it was assumed that the concrete slab was “simply-supported” all round. The selection of the load ratio is within the practical range of 0.3 to 0.7.

3 Experimental Results and Discussions

‘Failure’ point was identified as the time when either: (1) Full-depth cracks with the maximum crack width of about 10mm in the vicinity of the edge beams; or (2) There was a significant drop in the mechanical resistance, and it was not possible to maintain the load level (violation of criterion “R”) by continuous pumping.

3.1 Temperature developments

The temperature development of the slab is shown in Fig. 5(a). It can be observed that the air temperature consistently increased for all three tests. An acceptable discrepancy of 9.8% was found when comparing the temperature at 94min of heating, 999°C in P368-M1099 against 910°C in P486-M1099. The temperature at the bottom surface of the three slabs increased at the same rate up to 60min of heating. As temperature increased more concrete cracks developed resulting in significant heat loss. Thus, towards the end of the tests, the
corresponding temperature at the bottom surface and at the reinforcing mesh level showed some divergence. Temperature at the top surface was much lower than the furnace air temperature (due to the heat sink effect of concrete) with the maximum value of 150°C for all three tests.

Fig. 5(b) shows the temperature developments at the bottom flange of the main beams and the protected secondary edge beams. Temperature of the bottom flange of the edge beams increased quickly. Temperatures at the middle of the beam web and at the top flange were slightly smaller than that at the bottom flange, but they are not shown in Fig. 5(b). This is because the beam height of 131mm was small.

3.2 Slab displacements

Fig. 6 shows the average vertical deflection (measured by L1 and L2) at the slab centre against the mesh temperature \((w - T)\), and the temperature-time history \((T - t)\) of the three slabs. It should be noted that the bending stiffness about the major axis of PSB \((EI_{PSB})\) of P368-M1099 and of P486-M1099 had increased 1.71 and 2.26 times compared to that of P215-M1099.

P215-M1099 failed at a deflection of 124mm when the mesh temperature had reached 348°C at 72.2min of heating. The corresponding values were 118mm at 316°C at 74.6min, and 139mm at 412°C at 98.2min for P368-M1099 and P486-M1099, respectively. Fig. 6 indicates that as the stiffness of PSB increased, the slab deflection decreased. In P368-M1099 test, this trend was observed up to a mesh temperature of 280°C. From 280°C onwards, P368-M1099 experienced greater central deflections than P215-M1099, although the PSB of P368-M1099 had a greater stiffness relative to P215-M1099. This was because after 71min of heating (from 280°C onwards) one significant crack appeared above one main beam of P368-M1099. Consequently, composite action between the main beam and the concrete slab was weakened, and the beneficial effect of increasing the stiffness of PSB was lost. However, the
time at which the cracks in the vicinity of the edge beams occurred was unpredictable and
different for different specimens. Therefore, it was inexplicable why in the case of specimen
P368-M1099 the composite action had lost earlier than the other beams.

Fig. 7 shows the horizontal displacements of three slabs measured by L15 to L20 along
the slab edges. Positive values indicate inward displacements, while negative values indicate
outward displacements. Three important observations can be drawn from these figures.

Firstly, the horizontal displacement-time history of the slabs can be divided into three
stages. In the first stage, the slab edges moved outwards due to thermal expansion as
temperature increased. Up to 20 min of heating, the temperature was low because moisture in
the slab was gradually driven out. Therefore, the outward displacements of the slab edges in
two mutually-perpendicular horizontal directions were small, only about 2 mm. From 20 min
to about 50 min of heating, the outward displacements due to thermal expansion increased at a
greater rate with the maximum values of 8 mm, 9.7 mm and 9 mm for P215-M1099, P368-
M1099 and P486-M1099, respectively. In the second stage, after 50 min, when the slab
vertical deflection reached about $1h_s$ ($h_s$ is the slab thickness), tensile membrane action
(TMA) was mobilised. This can be recognised by the summation of outward displacements
due to thermal expansion and of inward displacements due to TMA, resulting in an almost
constant displacement rate in the second stage. In the third stage, when the slab experienced
large deflections, the slab edges moved inwards significantly. However, localised tensile
stresses in mesh reinforcement above the protected edge beams led to failure in these regions
with the maximum crack width in the slab attaining about 10 mm. When failure had been
identified, the furnace was turned off.

Secondly, the onset of TMA can be marked based on the horizontal displacement-time
history of the slabs. This was the time when the horizontal displacement rate reduced and was
almost constant (as in stage 2), resulting from the balance of outward and inward
displacements. Therefore, TMA was mobilised at 51min, 54min and 52min for P215-M1099, P368-M1099 and P486-M1099, respectively. These times are confirmed once again by the horizontal displacements of the columns (Section 3.3), and the development of crack patterns (Section 3.4).

Thirdly, it can be observed that the recorded displacements from L15, L16 and L17 measured along one slab edge, and those from L18, L19 and L20 measured along another slab edge were only slightly different. This indicates that the slab moved outwards or inwards in almost straight edges. Therefore, it can be concluded that the interior slab panels had been simulated accurately as postulated in Fig. 3(a).

### 3.3 Beam/Column behaviour

Fig. 8 shows a comparison of the mid-span vertical deflections of the edge beams, i.e. MB and PSB, against temperature at their bottom flanges. As temperatures developed, the beam deflection increased slowly due to thermal bowing. At a deflection of about 18mm for the main beams and 36mm for the secondary beams, when the temperature at the beam bottom flange reached 650°C, these edge beams almost lost all the fire protection and they experienced ‘run away’ deflections. It is shown that the deflection of MB was almost similar for all three slab assemblies because the main beam size was identical. As expected, the deflection of PSB decreased when the stiffness increased. After cooling, the specimens were taken out from the furnace. It was found that local buckling of the beam flanges had not occurred. This could be due to some room for thermal expansion through the flexible end plate connections.

Fig. 9(a) shows horizontal displacements of the supporting columns. Positive or negative values indicate outward or inward displacements of the column from the slab centre, respectively. L12 was used to measure the displacement along the secondary edge beam, and L13 was placed along the main beam (Fig. 4(b)). A similar trend in the displacement-time
history can be observed. The first stage began with the slow outward displacements in both
directions of the column due to thermal expansions of the edge beams as temperature
increased. The maximum value was 5mm for L12 of P215-M1099. When TMA was
mobilised in the slab, the column was pulled back to its original position by the edge beams.

The turning point between outward and inward displacements of the columns can be
considered as an indicator of the onset of TMA, which was mobilised after about 50min of
heating. It is noteworthy that this time roughly coincided with the time determined based on
the horizontal displacement-time history of the slab edges.

Fig. 9(b) shows the vertical displacements of the supporting columns. Positive or
negative values indicate downward or upward displacements recorded by L14, respectively.
It can be observed that all the specimens showed a similar behaviour. Initially, the columns
had a small axial deformation due to the applied load of 15.8kN/m². This UDL load (0.35
times the yield load at ambient temperature) only resulted in a small downward displacement.
The maximum value was only 1.5mm in P215-M1099. In the first stage, when heating
started, the column gradually moved upwards due to thermal expansion, and thus returned to
its original position. For instance, the column in P215-M1099 moved back to its original
position after 23.7min. As temperature increased, the column elongation increased as there
was no external restraint from thermal expansion. The maximum thermal elongation of the
columns was 7mm for P368-M1099 and P486-M1099.

In the second stage, when TMA was mobilised, the columns were pulled in due to
tensile forces in the edge beams. The softening of the columns also occurred due to heating.
Therefore, the columns moved downwards again, leading to an upward trend in the
displacement-time curves (Fig. 9(b)). The onset of TMA marked by this indicator was very
consistent compared with the other two indicators, i.e. horizontal displacements along the
slab edges (Fig. 7) and horizontal displacements of the column (Fig. 9(a)). The times
corresponding to the turning points of the columns are also shown in Fig. 9(b).

The deformed shapes of the protected steel columns after cooling are shown in Fig. 10, which clearly indicate that the protected columns were subjected to biaxial bending due to pulling-in of the edge beam ends in two mutually-perpendicular directions. The columns had withstood this effect and did not buckle.

3.4 Crack patterns & Failure modes

The development of crack pattern was observed carefully during the tests. The crack sequence shown by the numbers in Fig. 11 was very consistent in all three tests. Firstly, diagonal cracks near the beam-to-column joints (crack 1) appeared consecutively at the four slab corners. These cracks were due to biaxial bending of the slab outstand. At these corners, part of the outstand was in biaxial bending but restrained by the columns. Therefore, these cracks were consistently formed at an angle ranging from 30° to 45° to the slab edges.

Secondly, cracks appeared in the vicinity of the main beams (MB) (crack 2). These cracks were followed by diagonal cracks traversing from the slab corners to the columns (crack 3) since they were caused by in-plane restraint corner bolts (Fig. 1). As temperature increased, cracks along the protected secondary edge beams (crack 4) appeared. In P368-M1099, cracks above the unprotected interior secondary beams were observed (crack 4a). However, these cracks seemed to close up after natural cooling.

In P215-M1099 test, after 51min of heating, a compression ring began to form when the mesh temperature had reached 181°C at a deflection of 59mm or 1.03hₜ. In P368-M1099 and P486-M1099, compression ring began to form at a mesh temperature of 151°C and 185°C, after 54min and 52min of heating, respectively. The corresponding deflections were 54mm and 54mm, equal to 0.93hₜ and 0.98hₜ, respectively. Therefore, it can be concluded that TMA was mobilised at a slab deflection equal to 0.9 to 1.0 of the slab thickness,
irrespective of the stiffness of the protected edge beams. Tensile membrane mechanism consisted of radial tension in the central area of the slab and a peripheral compression ring.

The onset of TMA can be recognised by the appearance of diagonal cracks at the four corners (crack 5). The more obvious indicators are the turning points between the outward and inward displacements of the slab edges, as well as of the columns as explained in Sections 3.2 and 3.3. These three indicators occurred at very consistent times.

Severe cracks also appeared at the slab outstand (crack 6). These cracks were due to the bolts along the slab edges. As the slab deflected, these bolts were too stiff to deflect together with the slab causing cracking near the end of the bolts. When the slab experienced very large deflections, the cracks opened through the slab thickness and exposed reinforcement was fractured. However, this did not affect the test results because it happened after the failure had been identified.

In summary, the observed failure mode was due to fracture of reinforcement close to the protected edge beams. No global collapse occurred. No premature failure at the shear studs and at the connections was observed.

3.5 Enhancement factors

Table 2 shows that P486-M1099 (of which $I_{y,PSB}$ was increased 2.26 times compared to that of control specimen P215-M0119), experienced larger deflections, $2.53h_s$ compared to $2.17h_s$, an increase of 17%. However, only a slightly greater enhancement factor was obtained, 1.89 of P486-M1099 compared to 1.86 of P215-M1099. The enhancement factor is defined here as the ratio between the test load $p_{test}$ and the yield load at the failure mesh temperature $p_{y,0\theta}$. However, an identical increase of the maximum deflection was not found in P368-M1099. In this case, the maximum deflection was only $2.03h_s$ although its stiffness of PSB increased 1.71 times compared to that of P215-M0119. As explained above, this was due to weakening of the composite action between the main beam and the concrete slab after
min of heating.

Therefore, it can be concluded that an increase of the stiffness of protected secondary edge beams generally has positive effect on the slab behaviour. However, the composite action between the edge beams and the concrete slab plays a key role in mobilising this beneficial effect. Once the composite action has been diminished by cracks in the slab over the main or secondary edge beams, the slab would lose the benefit associated with greater stiffness of the secondary edge beams.

4 Numerical Simulation

4.1 Modelling technique

Modelling the thermal/structural behaviour of composite beam-slab systems consists of material and geometric nonlinearities, and large deformations. Abaqus/Explicit [19] was used to overcome numerical convergence difficulty. Sequentially coupled thermal-stress analysis procedure in Abaqus/Explicit was adopted because the thermal field is a major control for the stress analysis, but the thermal solution does not depend on the mechanical stress solution. Therefore, the thermal/structural behaviour of the composite floor systems can be obtained by using recorded temperature data without any need for thermal analysis. Concrete damaged plasticity model was used to simulate the concrete slab and layered rebar technique was adopted to model steel reinforcement. With regard to the concrete damaged plasticity model, for compression hardening, the compressive ultimate strains for concrete and the maximum concrete strain were taken from EN 1994-1-2 [20]. For tension stiffening, to avoid potential numerical problems, Abaqus enforces a lower limit on post-failure stress to be equal to one hundredth of the initial failure stress: \( \sigma_i \geq \sigma_{fo} / 100 \). Thus, if the concrete tensile stress is approximately equal to this limit, concrete can be considered as failed in tension. Steel coupon tests and concrete cylinder tests were conducted at ambient temperature to determine the material properties of steel and concrete, respectively. The stress-strain relationships and
thermal properties of the materials at high temperatures were obtained from EN 1994-1-2 [20].

The proposed numerical model takes into account of the steel beams, the concrete slab, and the reinforcing mesh as shown in Fig. 12. The vertical displacement at the column positions was assumed to be negligible. This assumption is reasonable because the maximum recorded vertical displacement at the column position was only 7mm in P486-M2110 (Fig. 9(b)), a mere 5% of the maximum slab deflection. Therefore, vertical restraint (U3 = 0) was imposed at the column locations. Vertical restraint along the edge outstand was used to model the rotational restraint beam system (Fig. 2); no springs were needed to model this system. As it was impossible to measure accurately horizontal reactions along the slab edges, the stiffness of inplane restraint beam system could not be determined. Therefore, two numerical cases were conducted for each specimen, viz. the first case with full horizontal restraint along the slab edges (Case 1), the second case without any horizontal restraint (Case 2).

Four-noded doubly-curved shell elements with reduced integration (S4R shell element) were used to discretize both the beams and the slab. The beam top flange and parts of the slab above the beams were tied together using surface-based contact interactions to simulate fully composite action between the beams and the slab. An offset between the two tied surfaces was adopted to avoid any overlap between the two median reference surfaces. This form of modelling implicitly assumed perfect bonding between the steel beam and the concrete slab.

4.2 Thermal response

Using the recorded temperatures, temperature development across the slab thickness can be identified without the need to conduct any heat transfer analysis. Since S4R shell elements were used to discretize the slab, it was not possible to input all the temperatures recorded at the slab bottom surface, reinforcing mesh and top surface into the model. Only the temperature at the slab bottom surface was input directly into the model to determine
thermal gradient of the slab. The recorded temperatures at the top and the bottom flanges and at the beam web can be input directly into the model.

In order to validate the structural response, the predicted temperatures were first compared with the measured temperatures as shown in Fig. 13. With a thermal gradient of 12°C/mm, a good correlation between the predicted and measured temperatures was obtained. There was only a small gap between the test result and the prediction at the reinforcing mesh. This is because during the tests, as temperature increased, concrete cracks were gradually developed resulting in heat loss. Consequently, the measured temperature at the reinforcing mesh was slightly lower than the prediction.

4.3 Deflection of the slabs and the interior beams

Comparisons between the predicted and the measured deflections of the slabs and the unprotected interior beams are plotted in Fig. 14. For the slab, the deflection is plotted against the mesh temperature; for the beam, the deflection is plotted against the temperature at the beam bottom flange. As explained before, since it was impossible to determine the stiffness of the inplane restraint system, two cases of boundary condition for the slabs were analysed. In Case 1, the slab edges were fully restrained horizontally. In Case 2, the slab edges did not have any horizontal restraint.

In reality, the measured mid-span slab deflection lies in between the predictions by Case 1 and Case 2 because the inplane restraint system provided only partial horizontal restraint for the slab. This trend was observed at the initial stage when the mesh temperature was below 150°C. However, when the mesh temperature increased from 150°C to 250°C, the slabs had greater deflections than the simulations. This may be caused by extensive cracking above the edge beams, which led to greater rotations of the slab about the edge beams compared to those predicted by the numerical models.
For P215-M1099 and P368-M1099, there is a gap between the test results and the prediction when the mesh temperature increased from 150°C to 250°C. This discrepancy can be explained by the differences between the measured and predicted temperatures in reinforcing mesh as shown in Fig. 13(a)-(b). Therefore, it can be concluded that the accuracy of a FE model in predicting the behaviour of composite slab-beam systems at large deflection is largely controlled by the accuracy of temperature predictions at the reinforcing mesh of the slab. Generally, the model predicts the slab behaviour well.

Fig. 14 also indicates that a good correlation between the predicted and measured mid-span deflections of the interior beams for the two cases, viz. with and without inplane restraint, was obtained.

4.4 Deflection of the protected edge beams

Comparisons between the test results and the predictions of the mid-span deflections of the protected edge beams for case 1 and case 2 are plotted in Fig. 15. It can be seen that case 1 gives slightly greater deflections of the edge beams than case 2 because the restraint forces induce greater deflections. However, the gap between the two sets of results is very small. Therefore, it can be concluded that horizontal restraint along the edges of the slab outstand has little effect on the vertical deflections of the edge beams.

It can be observed that the predictions agree well with the test results for the main and secondary edge beams up to 700°C. Beyond 700°C, although both the simulations and the tests showed a ‘run-away’ behaviour due to significant reduction in strength and stiffness of steel and concrete, the predictions give smaller deflections than the test results. This is possibly due to occurrences of concrete cracks above the edge beams. When these cracks had appeared and became more developed, the effective width of the concrete slab as a T-flange of the composite I beams reduced. Consequently, the composite action between the concrete slab and the steel beams weakened. In contrast, in the numerical models, the edge beams
would behave as T-flange composite beams throughout the fire durations, even though there was extensive cracking along the edge beams. This is a drawback of the numerical model.

4.5 Failure modes

The failure point could not be clearly captured in the FE models. Therefore, in order to make comparisons with the test results, failure of the composite floor systems is assumed when one or a combination of the following criteria is met, which are based on the BS 476 [21] provisions. It stipulates that failure of bending members (slab and beam) occurs if (a) the maximum central deflection exceeds \( L/20 \), or (b) deflection exceeds \( L/30 \), and the deflection rate exceeds \( L^2/9000h_s \), where \( L \) and \( h_s \) are the clear span and slab thickness, respectively. Checked against the test results, the first deflection limit \( (L/20) \) is the most critical for the slabs. Besides, due to the scale effect and non-standard fire condition, the deflection rate criterion could not be applied. For the slab, the excessive deflection (those exceed \( L/20 \)) is calculated to be 113mm. For the main and protected secondary beams, the corresponding values are 105mm and 112mm, respectively.

Table 3 summarises the measured and predicted temperatures of the slabs at the ‘so-called’ failure point of 113mm. Temperatures of reinforcing mesh in the models at the ‘assumed’ failure point are compared with those from the experiment. A good correlation between the measured and predicted results is obtained with the test/prediction ratio lying between 0.83 and 1.0.

Table 4 shows the comparisons between the measured and predicted deflections of the edge beams at the so-called ‘failure’ point. The beam deflections from the simulations are compared to those from the test results at a similar deflection of the slab, 113mm. It can be seen that the predictions for the main beam (MB) are very good with the test/prediction ratio lying between 0.86 and 0.96, while those for the protected secondary beam (PSB) are poorer (1.21 to 1.25). This is caused by occurrences of concrete cracks above the edge beams. As
explained above, when these cracks had appeared and became more developed, the effective
width of the concrete slab as a T-flange of the composite I beams reduced. However, in the
numerical models, the edge beams would behave as T-flange composite beams throughout
the fire durations, even though there was extensive cracking along the edge beams. Generally,
the model predicts the beam deflections reasonably well.

5 Conclusions

This paper presents novel experimental results and observations on three composite
beam-slab systems tested under fire conditions with the aim to study the effect of bending
stiffness of the protected secondary edge beams on the behaviour of floor assemblies. The
proposed numerical models have been developed and validated with the test results. Based on
the test results and the numerical investigations, the following conclusions can be drawn:

(1) As the stiffness of the protected secondary edge beams increased, the slab deflection
initially decreased and failure of the slab occurred later. However, composite action
between the edge beams and the concrete slab plays a key role in mobilising this
beneficial effect. Once the composite action had been weakened by cracks in the slab
over the main or secondary edge beams, the benefit associated with a greater stiffness
of the edge beams was lost.

(2) Tensile membrane action was mobilised at a deflection equal to 0.9 to 1.0 of the slab
thickness irrespective of bending stiffness of the edge beams. The commencement of
tensile membrane stage was marked by one of three indicators: (a) concrete cracks
which formed a peripheral compressive ring on top of the slabs; (b) horizontal
displacements along of the slab edges; and (c) horizontal and vertical displacements
of the columns. In all the tests conducted, these indicators were very consistent in
terms of occurrence times.
(3) All three specimens failed due to severe cracks appearing in the vicinity of the protected edge beams, resulting in fracture of reinforcement at these regions.

(4) The numerical model was able to predict accurately the thermal/structural behaviour of composite slab-beam systems under fire conditions. However, the model assumed perfect bonding between the steel beam and the concrete slab. Therefore, it could not simulate accurately deterioration of composite action between the steel downstand beams and the slab as temperature increases and as cracking takes place. This is a drawback of the numerical model.

ACKNOWLEDGMENT

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<th>Denote</th>
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<th>Width ( b_f ) (mm)</th>
<th>Thickness (mm)</th>
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Table 2 Summary of the test results

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*$\theta_t$: temperature at slab top surface; $\theta_m$: temperature at reinforcing mesh; $\theta_b$: temperature at slab bottom surface;
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