Behaviour of composite beam-slab floor systems under fire conditions

Nguyen, Tuan Trung

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**NANYANG
TECHNOLOGICAL
UNIVERSITY**

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FLOOR SYSTEMS
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**NGUYEN TUAN TRUNG
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
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NGUYEN TUAN TRUNG

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

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ABSTRACT

Due to some fire incidents widely known as the Basingstoke and the Broadgate Fires, composite slabs consisting of steel beams and lightly reinforced concrete slabs with steel decking showed an inherent load-carrying capacity which has not been taken into account by any building code. Previous research has shown that fire resistance of composite slabs can be considerably enhanced if tensile membrane action (TMA) can be mobilised at large displacements. However, there are some gaps in the understanding of membrane behaviour of composite slab-beam systems in fire.

The objectives of the research work are to: (1) Quantify the physical behaviour of slabs incorporating realistic behaviour of fire-protected composite edge beams by conducting two test series on composite beam-slab floor systems (CB-S systems) under fire conditions; (2) Develop the nonlinear finite element models which can predict the behaviour of CB-S systems in fire; (3) Propose a semi-analytical approach to estimate the load-bearing capacity of CB-S systems in fire; (4) Propose design recommendations on the current design approach (the Bailey-BRE method) using TMA concept.

Based on preliminary numerical analyses using the explicit dynamic solver in ABAQUS, an experimental programme was proposed. Two test series with eight CB-S systems in total were designed in accordance with Eurocode standards and tested under fire conditions. The experimental objectives were to investigate the effects of unprotected interior beams, rotational edge restraint and stiffness of the edge beams on the development of TMA in concrete slabs.

The experimental observations are of significance. *Firstly*, fracture of reinforcement above the protected edge beams, rather than in the central region, was the final collapse mode of CB-S systems. *Secondly*, it is possible to omit fire protection for interior secondary beams, which leads to cost saving of fire-protection materials without compromising safety of the systems. *Thirdly*, TMA was mobilised at a

deflection equal to approximately 0.9 to 1.0 of the slab depth irrespective of the presence of interior beams, or bending stiffness of the edge beams.

Using the results and observations from the experiment, nonlinear finite element models using ABAQUS/Explicit have been proposed. These numerical models were validated extensively with the author's tests and with small- and full- scale tests in the literature. The models were capable of predicting accurately the structural behaviour of CB-S systems subjected to fire and provided insight into the behaviour of the systems.

A semi-analytical model was developed to estimate the enhancement of the load-bearing capacity of composite slab-beam systems under fire conditions due to TMA. Unlike other current approaches, this model also considers the vertical deflections of edge beams. The model has been shown to give conservative and more accurate predictions compared to the Bailey-BRE method. It also gives information of the edge beams which the Bailey-BRE method does not provide.

Finally, design recommendations are proposed in order to facilitate the use of TMA concept in practice. Most importantly, in order that TMA can be mobilised in the systems without compromising the safety, special stipulations for structural members shall be followed closely in the design process, such as verification of the edge beams, and calculation concept of unprotected interior beams at failure temperature, i.e. considering the unprotected interior beams as composite or steel beams, when calculating the contribution of the interior beams towards the total load-bearing capacity of the floor system.

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NOTATIONS

A, B, C, D	parameters to calculate the enhancement factor in the Bailey-BRE model
$\overline{A}, \overline{B}, \overline{C}, \overline{D}, \overline{E}$	parameters to calculate the enhancement factors in the proposed model
$E_{a,\theta}$	elastic modulus of structural steel at temperature θ
E_a	elastic modulus of structural steel at ambient temperature
E_s	elastic modulus of reinforcing steel at ambient temperature
E_{cm}	secant modulus of elasticity of concrete
EI_y	bending stiffness of steel beam about the major axis
EI_z	bending stiffness of steel beam about the minor axis
GI_t	torsional rigidity of steel beam
I_{yPSB}	second moment of area about the major axis of protected secondary edge beam
I_{yMB}	second moment of area about the major axis of protected main edge beam
T_{2b1} and T_{1b1}	respective temperatures at the bottom and the top surfaces of the edge beam
$T_{,zb1}$ and $T_{,zb2}$	thermal gradient over the total depth of beams 1 and 2, respectively
a	aspect ratio of the slab (L/l)
b_f	width of beam flange
d_x	effective depth of slab according to short span
d_y	effective depth of slab according to long span
e	overall enhancement factor proposed by Bailey
e^*	overall enhancement factor proposed by the author
e_{1m}, e_{2m}	contribution of membrane forces to the load-bearing capacity

e_{1b}, e_{2b}	factors taking into account of the effect of membrane forces on the bending resistance due to the presence of axial force
e_1, e_2	enhancement factors of elements 1 and 2 calculated with the rigid edges
e_1^*, e_2^*	enhancement factors of elements 1 and 2 calculated with deformed edges
k, b	parameters to determine the in-plane stress distribution
$f_{y,\theta}$	yield strength of steel at elevated temperatures
$f_{p,\theta}$	proportional limit of steel at temperature θ
f_y	yield strength of steel at ambient temperature
f_u	ultimate strength of steel at ambient temperature
$f_{c,\theta}$	concrete ultimate stress at temperature θ
f_{cm}	mean value of concrete cylinder strength at ambient temperature
f_{ctm}	mean tensile strength of concrete
f_{ck}	characteristic concrete cylinder strength at ambient temperature
$(g_0)_1$	parameter defining the flexural stress block in the short span
$(g_0)_2$	parameter defining the flexural stress block in long span
h_s	slab thickness
h_b	total depth of composite beam
h_{sb}	depth of steel beam
h_{b1}	total depth of the composite beam 1
h_{sb1}	depth of the steel beam 1
l and L	shorter and longer spans of the slab panel
$p_{y,\theta}$	yield-load of the slab at temperature θ
$q_{b,\theta}$	load-bearing capacity of the interior beams at temperature θ
$q_{s,\theta}$	load-bearing capacity of the slab enhanced by TMA at temperature θ
$q_{t,\theta}$	total load-bearing capacity of the slab-beam system at temperature θ

t_f	thickness of beam flange
t_w	thickness of beam web
$w_{zx}(x)$	deformed shape of beam 1
$w_{zy}(y)$	deformed shape of beam 2
w_m	absolute slab deflection
w_r	relative slab deflection
α	coefficient of thermal expansion (12×10^{-6} for normal weight concrete)
$\alpha_1, \beta_1, \gamma_1, \delta_1$	parameters to calculate the enhancement factors in the proposed model
$\alpha_2, \beta_2, \gamma_2$	parameters to calculate the enhancement factors in the proposed model
α_3, β_3	parameters to calculate the enhancement factors in the proposed model
$\varepsilon_{c1,\theta}$	concrete ultimate strain at temperature θ
$\varepsilon_{p,\theta}$	strain at the proportional limit at temperature θ
$\varepsilon_{y,\theta}$	yield strain at temperature θ
$\varepsilon_{t,\theta}$	limiting strain for yield strength at temperature θ
$\varepsilon_{u,\theta}$	ultimate strain at temperature θ
μ	ratio of the yield moment capacity of the slab in orthogonal directions

CHAPTER 1 INTRODUCTION

1.1 Background

Over the past two decades, composite steel deck-concrete slab systems have been widely used in modern office buildings, since this type of slab system can provide considerable advantages in terms of ease of construction, reduction of site work and cost. A typical composite steel framed building consists of steel columns and primary beams spanning between the columns, with secondary steel beams spanning between the primary beams as shown in **Fig. 1.1(a)**. These members in turn support the concrete slab cast on profiled steel sheeting (**Fig. 1.1(b)**). However, compared to reinforced concrete structures, steel decks and supporting steel beams perform rather poorly under fire conditions because both concrete and steel lose strength and stiffness in a fire. It's the thermal conductivity that is important.

a) Composite building

b) Typical structural system

Fig. 1.1 Typical composite building

In recognition of the inherent disadvantages of steel material in fire, a great amount of research has been conducted to determine new protective materials and design methods for fire protection of steel structures. The traditional means of achieving

specified periods of fire resistance for steel framed buildings is to apply *passive fire protection* to structural elements using protective materials. However, this form of protection is costly due to additional expenditure and construction time. Thus, it requires alternative methods to reduce fire protection cost.

Besides, most of the current fire safety design codes rely on tests conducted on individual members where fire resistance is based on *standard fire tests*. These design codes are conservative since they ignore structural interactions under standard fire exposure. Therefore, over the recent years, there has been a steady trend to study more realistic approaches for steel and composite structures to take account of the beneficial effect of structural interactions under fire conditions.

1.2 Fire Resistance Assessment

Fire resistance analysis plays an important part in any fire safety design. The aim is to ensure that the design fire resistance must be greater than the fire severity. The most common method of assessing the fire resistance of structural members is by conducting standard fire resistance tests. However, such tests should only be conducted when necessary since they are very expensive. Due to the availability of published research works on isolated members such as beams, columns, walls and slabs, it is now possible to determine the fire resistance using calculation methods. Fire resistance calculation consists of two main considerations, namely, thermal and structural response analyses.

1.2.1 Fire resistance by testing

Testing is the traditional means to evaluate the fire performance of individual structural members. Normally, *standard fire resistance tests* are used. Unlike compartment fires, standard fire exposures are controlled by prescribed temperature-time relationships in the furnace, which are defined as *the standard fire curve*. Using the standard fire curve, the assessment is based on *fire limit state* which includes load-bearing capacity (*R*), insulation (*I*) and integrity (*E*). During the test, these failure criteria are monitored. If any of them is violated, the time with respect to the

standard fire curve is measured and this forms the fire resistance for the tested element.

There are several standard fire curves adopted in different codes. The most commonly used fire curve is based on ISO 834 (ISO 1975) as shown in **Fig. 1.2**. The fire temperature, T_g (°C), is defined by:

$$T_g = 345 \log_{10}(8t + 1) + T_0 \quad (1.1)$$

T_0 : initial temperature (usually taken as 20°C); T_g : gas temperature; t : time (min).

Fig. 1.2 ISO 834 Fire Curve

Although the concept of fire resistance is widely accepted in practice, it does not realistically represent the actual behaviour of a member in a fire. The temperature-time curve used is unrealistic since it is not representative of a compartment fire which includes a more accurate description of the fire load inside the compartment. Besides, the test only considers individual elements and is not suitable for testing sub-structures that would include interactions between the elements and their boundary conditions. This aspect is extremely important as it has been shown that only a small amount of adjacent restraint has a significant beneficial effect on the structural response of a single element to fire (Usmani *et al.* 2001). However, the fire resistance test is still widely accepted because extensive data have been established based on the ISO 834 fire curve. Besides, it provides a common basis of comparison for isolated members. It is also very expensive to retest these specimens to a new fire curve.

1.2.2 Fire resistance by Design codes

British Standards

BS 5950 Part 8 provides design recommendations for determining the fire resistance of steel structures. It considers the fire limit state to be an accidental limit state, in which load factors and material strength factors are given. The most common method of achieving the specified fire resistance is the application of passive fire protection. The required thickness of fire protection is derived from the section factor, defined by the ratio of the heated perimeter to the gross cross-sectional area. The thickness of fire protection is then chosen based on the section factor by reference to 'The Yellow Book' (ASFP *et al.* 2004), or alternatively, calculated according to the formula given in the code.

Calculation methods in BS 5950 Part 8 include the limiting temperature method or the moment capacity method. Both simple but effective procedures use the concept of load ratio, defined by the ratio of the load carried during a fire to the load capacity at ambient temperature. The limiting temperature method can be applied to columns, tension members and beams. Based on the load ratio, the limiting temperature is compared with the design temperature to assess the need for passive fire protection. An alternative method is to use the moment capacity method, which can be applied to beams. The moment capacity is calculated by means of known temperature distribution of the element using a strength reduction factor. If the moment capacity does not exceed that applied at the fire limit state then the beam can be left unprotected.

Composite slabs are also included in the code by the use of simple look-up tables. These tables contain minimum dimensions of the element and minimum cover to the main reinforcement for common periods of fire resistance.

Although BS 5950 Part 8 was the first design code to provide a simplified fire-resistance design calculation method for structural members, it only allows the use of the standard fire curve. Besides, it does not consider structural continuity and

interaction with surrounding unheated structures. Furthermore, the use of the calculation methods only allows members to carry load by bending action. In general, these provisions have shown to provide an acceptable level of safety based on experimental tests in fire conditions.

Eurocode

The Eurocode includes four parts for designing concrete, steel and steel-concrete composite structures under fire conditions. The design methods in the Eurocode are more general than those in the British Standard. The fire design methodology in the Eurocode allows engineers to have greater flexibility in their approach by introducing four methods, namely, global structure analysis, analysis of sub-structures, analysis of isolated members and testing method. These methods give engineers the opportunity to use advanced numerical models or alternative design methods.

This rather complex assessment process can be simplified into a three-phase procedure consisting of the characterization of fire load in the compartment, determination of the temperature distribution within the structure and an assessment of the structural response to the fire.

The temperature-time curves in the Eurocode may either be nominal fire curves or physics-based fire models. Typical nominal curves include the ‘standard’ fire curve, such as ISO 834 and BS EN 1363-1, while physics-based natural fire models consist of a parametric approach, time-equivalent method and other advanced fire calculations.

The Eurocodes also provide comprehensive material models for concrete and steel at elevated temperatures. Thermal properties of materials are also given in the codes. These properties are adopted in this research and are presented in **Section 2.2**.

1.3 Cardington Fire Tests on a Composite Steel-framed Building

Since the current design codes are based on the results of standard fire tests, structural interactions are usually ignored in design. In recognition of this limitation, to obtain a better understanding of the real structural behaviour of modern composite steel-framed buildings in fire, a series of six full-scale tests were conducted at Cardington by British Steel and the Building Research Establishment (BRE). The test building was a steel-framed composite construction, using in-situ concrete slabs supported by steel decking and in composite action with supporting steel beams (**Fig. 1.3**). These tests clearly demonstrated that such buildings have superior fire performance not considered before in the building codes.

Fig. 1.3 The Cardington Test Building (Newman et al. 2006)

The most important lesson of the Cardington fire tests is the good performance of floor slabs. Although the underside of the trapezoidal composite decks was unprotected, the composite floor slabs showed no sign of imminent collapse. It was therefore suggested that the floor load was resisted by a different load-carrying

mechanism than flexural bending that was commonly assumed in conventional design. It was also observed that unprotected secondary steel beams suffered a significant loss in strength and stiffness, and the steel deck was debonded from the lightly reinforced concrete (LRC) slab (Bailey and Moore 2000a; Newman *et al.* 2006). Thus the principal load-carrying structural component under fire conditions was the lightly reinforced concrete slab which experienced large deflections. The load-carrying mechanism has since been identified as tensile membrane action (Wang 1996).

After the Cardington fire tests, research studies on the behaviour of composite floor slabs under fire conditions have intensified. A number of experimental, numerical and analytical studies on tensile membrane action of composite floor slabs at elevated temperatures have been conducted (Huang *et al.* 1999; Bailey and Moore 2000a; Usmani and Cameron 2004). A design guide had been released in the UK, *SCI Publication P288* (Newman *et al.* 2006), in order to reduce fire protection cost for composite floor assemblies by taking account of tensile membrane action (TMA). However, an extensive review discussed in **Chapter 2** shows that there are still some technical gaps in the understanding of membrane action. These include the following:

- Effect of perimeter beam deflections on the development of tensile membrane action;
- Effect of stiffness and strength of fire-protected edge beams on the mobilisation of tensile membrane action;
- Effect of rotation restraint of concrete slabs on tensile membrane action;
- Effect of unprotected interior secondary beams on the behaviour of the composite floor systems;

Therefore, to obtain a better understanding of the behaviour of composite slabs with fire-protected edge beams in fire, a study has been conducted in Nanyang Technological University (NTU), Singapore. The results will be very useful in identifying weaknesses of the current design methods. It would be possible to

improve these methods in assessing the load-carrying capacity of slab-beam composite floor systems in fire.

The following terminology has been adopted in this thesis:

- *Main beam* – the beam spanning between panel corners and parallel to the span direction of the one-way composite floor and on the column grid.
- *Secondary edge beam* – the beam spanning between panel corners and perpendicular to the span direction of the one-way composite floor.
- *Interior beam* – the beam spanning in the same direction as the secondary edge beam but supported at points along the length of the main beams and are off the column grid.

1.4 Objectives and Scope of work

This research focuses on the structural behaviour of composite beam-slab floor systems (CB-S systems) subjected to elevated temperatures with the following objectives:

1. To obtain a physical understanding of the structural behaviour of CB-S systems under fire conditions based on experimental investigation;
2. To propose validated nonlinear finite element models which are capable of predicting structural and thermal behaviour of CB-S systems in fire;
3. To propose a semi-analytical model to estimate the load-bearing capacity of CB-S systems in fire taking into account of deflections of supporting edge beams on tensile membrane action.
4. To suggest design recommendation on the current design guides using TMA concept, SCI Publications P288 and P390.

Since the behaviour of the floor assemblies in fire is very complex, in order to achieve the above-mentioned objectives, the following simplifications are made:

1. Edge beams are connected to the columns which are assumed to have sufficient bending strength. As a result, the supporting columns will be the last members to fail (if at all);

2. At elevated temperatures, steel decking suffers a significant loss in strength and stiffness. Therefore, it is reasonable to assume that the contribution of steel decking to the load-bearing capacity of composite slab is ignored under fire conditions;
3. One-fourth scale specimens were designed and tested in this experimental programme. The size effect is assumed negligible;
4. Only structural behaviour of members is investigated without considering the behaviour of connections;
5. The research is focused on structural behaviour, rather than thermal behaviour of the members. Hence, the “fire resistance” from the tests cannot be compared with the standard fire tests using ISO 834 fire curve.

With regard to item 3, small-scale tests at elevated temperatures can result in unrealistic temperature distributions in the beams and slabs. Thus, this study cannot give any advice on fire resistance time of the floor systems. However, the experimental results do provide basic information on the membrane behaviour in fire. They also allow analytical methods and numerical models to be validated. In future work, realistic temperature distributions, appropriate to full-scale slab beam floor systems, can then be incorporated into the validated numerical models. On the other hand, for the structural components which their behaviour is controlled by bending not shear, such as reinforced concrete thin slabs, the scale effect would have a little effect on the behaviour.

1.5 Layout of thesis

The thesis is organised into nine chapters. This chapter introduces some basic concepts for fire safety design, such as the standard fire curves and fire resistance assessments. The problem statements, the objectives and the scope of work are defined.

Chapter 2 provides an extensive review of previous studies related to the behaviour of composite slabs in fire. The review covers previous experimental works, analytical approaches and numerical investigations on tensile membrane action.

Thermal and mechanical properties of concrete and steel at elevated temperatures are also discussed.

In **Chapter 3**, preliminary numerical studies using ABAQUS/Explicit have been conducted to determine the keys parameters which have significant effects on the behaviour of CB-S systems in fire. The proposed numerical models are first validated by a number of published test data, and then a parametric study is conducted.

Chapter 4 presents preparatory works for the experiment programme in this thesis including the setup of a new electric furnace, and the design for test specimens. Details of test specimens, test set-up, loading arrangements are presented here.

Experimental results and observations of two test series are discussed in **Chapters 5** and **6**. After validated with the test results, numerical investigations are conducted in order to provide insight into the behaviour of the floor assemblies.

Chapter 7 presents a semi-analytical approach which can be used to predict the load-bearing capacity of CB-S systems enhanced by TMA under fire conditions. The proposed model can consider the deflections of protected edge beams and their effect on TMA capacity. Comparisons between the proposed model, the current design approach using TMA (the Bailey-BRE method), and the test results are introduced in this chapter.

Design recommendations on the current design guides using TMA concept, SCI Publications P288 and P390, are discussed in **Chapter 8**.

Finally, conclusions and future works are discussed in **Chapter 9**.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

For composite steel-framed buildings, the most common and adequate method for fire protection is by applying insulating materials, such as concrete spray or intumescent paint. This approach provides sufficient strength for members within specified fire duration by limiting the temperature. This approach is simple but costly due to additional expenditure and construction time. Recently, there is a trend to design composite steel framed buildings under fire conditions by using advanced calculation methods, i.e. *performance based approach*. One of these methods is mentioned in Newman *et al.* (2006), which allows a reduction in the fire protection cost for interior secondary beams. The concept relies on tensile membrane action (TMA) developed in composite slab panels at large deflections.

Therefore, an extensive literature review relevant to the behaviour of composite slabs under fire conditions has been conducted to define the objectives and the scope of work. The temperature-dependent material properties which are critical to structural behaviour in fire are discussed first. This is followed by a discussion of the behaviour of slab and slab-beam composite floor systems at elevated temperatures. The chapter ends with a critical review of the most important parameters that govern tensile membrane action in slabs and composite floor assemblies.

2.2 Material Properties at Elevated Temperatures

Constituent material models at elevated temperatures are crucial for the conduct of accurate structural responses in fire. Thus, a review of previous research works in modelling of mechanical properties of steel and concrete was conducted. Most of the works have been adopted in EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2. Furthermore, thermal properties of materials are also introduced. Thermal properties include thermal conductivity, specific heat, mass loss, and thermal strain. Among the

thermal properties, thermal strain is the only concern here since it is associated with structural responses.

All of the aforementioned properties are temperature-dependent and are strongly influenced by the composition and characteristics of materials. The increase and distribution of temperature in a structural member are deeply affected by the thermal properties, whereas the mechanical properties determine the strength degradation as well as the stiffness deterioration of materials.

2.2.1 Material behaviour at elevated temperatures and testing regimes

Material properties of steel and concrete at elevated temperatures are considerably different from those at ambient temperature. Generally, with increasing temperature, the strength and stiffness of the materials decrease while ductility increases. At time t in a fire the total strain consists of four components:

$$\varepsilon = \varepsilon_{th}(\theta) + \varepsilon_{\sigma}(\sigma, \theta) + \varepsilon_{cr}(\sigma, \theta, t) + \varepsilon_{tr}(\sigma, \theta) \quad (2.1)$$

Thermal strain ε_{th} is the strain induced by thermal expansion. Depending on the boundary conditions, i.e. restraints and the temperature distribution within a member, this value can induce high stresses. Mechanical strain ε_{σ} is caused by mechanical stresses and can be determined by the stress-strain-temperature relationship of a particular material. Creep strain ε_{cr} is the long-term strain acted upon by a constant load. The fourth component is transient strain ε_{tr} which only develops under compressive stresses in heated concrete structures.

Eq. (2.1) explicitly includes the creep effect. There is another approach, namely, a pragmatic model, which implicitly includes the effect of creep. In fact, Twilt (1986) opined that it is not necessary to use a complicated creep model under standard fire conditions because a pragmatic model is sufficient. His investigation implied that in terms of fire resistance the creep effect is negligible. In view of other more important uncertainties involved, such degree of accuracy is meaningless. Therefore, the pragmatic model is normally adopted for simulating the constitutive models at elevated temperatures.

To develop a constitutive model for steel and concrete at elevated temperatures, a large number of test data involving stress and deformation characteristics at various temperatures are needed. There are two test regimes to obtain these data, namely, *isothermal* (or steady-state) tests and *anisothermal* (or transient-state) tests. In the *isothermal* tests, a specimen is heated up to a constant temperature before mechanical load is applied. The strain measured before any load application corresponds to thermal strain. In the *transient-state* tests, a specimen is subjected to a constant load while the temperature is increased according to a nominal fire curve. The total strains measured consist of mechanical, thermal and creep strains. Thermal strain can only be measured by using unloaded specimens heated at the same rate.

2.2.2 Steel Properties at Elevated Temperatures

2.2.2.1 Stress-strain-temperature relationship

The stress-strain relationships and thermal properties of steel specified in EN 1993-1-2 (EC3) (2005b) are both used in this research due to their widespread acceptance. The EC3 model adopted the bilinear-elliptic model based on the *transient-state test* data from Kirby and Preston (1986). **Fig. 2.1** shows a typical stress-strain curve adopted in EC3. The curve consists of a straight line for the initial response, followed by an elliptical relationship, then a plateau and finally a declining line down to the ultimate strain ($\varepsilon_{u,\theta}$). This value is assumed to be temperature-independent. The values of $\varepsilon_{t,\theta}$ and $\varepsilon_{u,\theta}$ are 0.15 and 0.2, respectively.

Fig. 2.1 Stress-strain relationship for carbon steel at elevated temperatures

Table 2.1 lists the stress-strain equations of carbon steel material for various temperatures.

Table 2.1 Mathematical formulations of stress-strain relationship for carbon steel

Strain range	Stress $\sigma_a(\theta_a)$	Tangent modulus
$\varepsilon \leq \varepsilon_{p,\theta}$	$\varepsilon E_{a,\theta}$	$E_{a,\theta}$
$\varepsilon_{p,\theta} < \varepsilon < \varepsilon_{y,\theta}$	$f_{p,\theta} - c + \frac{b}{a} \sqrt{a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2}$	$\frac{b(\varepsilon_{y,\theta} - \varepsilon)}{a \sqrt{a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2}}$
$\varepsilon_{y,\theta} \leq \varepsilon \leq \varepsilon_{t,\theta}$	$f_{y,\theta}$	0
$\varepsilon_{t,\theta} < \varepsilon < \varepsilon_{u,\theta}$	$f_{y,\theta} \left[1 - \frac{(\varepsilon - \varepsilon_{t,\theta})}{(\varepsilon_{u,\theta} - \varepsilon_{t,\theta})} \right]$	—
$\varepsilon = \varepsilon_{u,\theta}$	0.00	—
Parameters	$\varepsilon_{p,\theta} = f_{p,\theta} / E_{a,\theta}$ $\varepsilon_{y,\theta} = 0.02$ $\varepsilon_{t,\theta} = 0.15$ $\varepsilon_{u,\theta} = 0.20$	
Functions	$a^2 = (\varepsilon_{y,\theta} - \varepsilon_{p,\theta})(\varepsilon_{y,\theta} - \varepsilon_{p,\theta} + c / E_{a,\theta})$ $b^2 = c(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} + c^2$ $c = \frac{(f_{y,\theta} - f_{p,\theta})^2}{\left[(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} - 2(f_{y,\theta} - f_{p,\theta}) \right]}$	

where: $f_{y,\theta}$: effective yield strength at elevated temperatures; $f_{p,\theta}$: proportional limit at temperature θ ; $E_{a,\theta}$: elastic modulus at temperature θ ; E_a : elastic modulus at ambient temperature; $\varepsilon_{p,\theta}$: the strain at the proportional limit at temperature θ ; $\varepsilon_{y,\theta}$: the yield strain at temperature θ ; $\varepsilon_{t,\theta}$: the limiting strain for yield strength at temperature θ ; $\varepsilon_{u,\theta}$: the ultimate strain at temperature θ .

Table 2.2 shows the associated reduction factors for effective yield strength $f_{y,\theta}$, proportional limit $f_{p,\theta}$ and tangent modulus $E_{a,\theta}$.

Table 2.2 Reduction factors for EC3 stress-strain relationships of steel

Steel temperature θ_a ($^{\circ}\text{C}$)	Reduction factor for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$	Reduction factor for proportional limit $k_{p,\theta} = f_{p,\theta}/f_y$	Reduction factor of tangent modulus $k_{E,\theta} = E_{a,\theta}/E_a$
20	1.000	1.000	1.000
100	1.000	1.000	1.000
200	1.000	0.807	0.900
300	1.000	0.613	0.800
400	1.000	0.420	0.700
500	0.780	0.360	0.600
600	0.470	0.180	0.310
700	0.230	0.075	0.130
800	0.110	0.050	0.090
900	0.060	0.0375	0.0675
1000	0.040	0.025	0.0450
1100	0.020	0.0125	0.0225
1200	0.000	0.000	0.000

As can be seen in **Table 2.2**, the mechanical properties of steel remain unchanged at temperature 100°C and only slightly decrease at temperature up to 200°C . However, these properties significantly reduce above 500°C . Additionally, the elastic modulus of steel reduces more rapidly than the strength. **Fig. 2.2** shows the EC3 normalized stress-strain-temperature relationships of hot-rolled steel as an illustration of these trends. It should be noted that from 100°C to 400°C , the reduction factor for the yield strength equals to 1.0. Therefore, the stress-strain curves for the temperature ranging from 100°C to 400°C are overlapped in **Fig. 2.2**.

Fig. 2.2 Engineering stress-strain curve of hot-rolled steel using EC3 model

It is note-worthy that EN 1993-1-2 provides the constitutive model for steel in the form of engineering stress-strain curves. To adopt this model in advanced finite element programs, such as ABAQUS, an engineering stress-strain curve needs to be converted to a true stress-strain curve. The relationship between these two curves is introduced in **Section 3.3**.

2.2.2.2 Thermal expansion

Thermal strains are measured on un-loaded specimens under heating and can be determined from temperature θ using commonly accepted relationship. EN 1993-1-2 gives the values of thermal strain in three different ranges of temperature:

$$\varepsilon_{th}(\theta) = -2.416 \times 10^{-4} + 1.2 \times 10^{-5} \theta + 0.4 \times 10^{-8} \theta_a^2 \text{ for } 20^\circ\text{C} \leq \theta < 750^\circ\text{C} \quad (2.2)$$

$$\varepsilon_{th}(\theta) = 1.1 \times 10^{-2} \quad \text{for } 750^\circ\text{C} \leq \theta \leq 860^\circ\text{C} \quad (2.3)$$

$$\varepsilon_{th}(\theta) = -6.2 \times 10^{-3} + 2 \times 10^{-5} \theta \quad \text{for } 860^\circ\text{C} < \theta \leq 1200^\circ\text{C} \quad (2.4)$$

2.2.3 Concrete Properties at Elevated Temperatures

The concrete properties at elevated temperatures are more variable and complex than the steel properties. Only some properties that are relevant to this study are discussed, namely, stress-strain relationship, modulus of elasticity, thermal expansion and spalling.

2.2.3.1 Stress-strain relationship

Due to composite nature of concrete, its stress-strain relationship is highly complex because it behaves very differently in compression and in tension. Failure occurs by crushing in compression but cracking in tension, which leads to a discontinuous stress-strain curve. The concrete tensile strength is typically taken as 10% of the compressive strength (Schneider 1986a). There are a number of factors affecting the concrete strength (tension and compression) such as the amount of cement paste, the type and size of aggregate, water/cement ratio and concrete age. These factors cause its behaviour in fire to be less predictable than that of steel.

EN 1992-1-2 (2004a) provides a stress-strain relationship of concrete under compression at elevated temperatures, which consists of ascending and descending parts (**Fig. 2.3**). Eq. (2.5) shows the equation for the ascending part.

$$\sigma(\theta) = \frac{3\left(\varepsilon / \varepsilon_{c1,\theta}\right)}{\left[2 + \left(\varepsilon / \varepsilon_{c1,\theta}\right)^3\right]} f_{c,\theta} \quad (2.5)$$

where $\sigma(\theta)$, ε , $f_{c,\theta}$, $\varepsilon_{c1,\theta}$ are the concrete stress, strain, ultimate stress and strain at a temperature θ , respectively.

Fig. 2.3 Stress-strain relationship of concrete at elevated temperatures

Table 2.3 shows the reduction in strength $k_c(\theta) = f_{c,\theta} / f_{ck}$, $\varepsilon_{c1,\theta}$, and $\varepsilon_{cu1,\theta}$ for siliceous and calcareous normal weight concretes (NWC) according to EN 1992-1-2. As temperature increases, the peak compressive strength $f_{c,\theta}$ reduces whereas the corresponding strain increases. **Fig. 2.4** shows the normalized stress-strain relationship for concrete at elevated temperatures.

Table 2.3 Strength reduction factors and strain limits of NWC at elevated temperature

Temperature $\theta(^{\circ}\text{C})$	Siliceous NWC			Calcareous NWC		
	$f_{c,\theta}/f_{ck}$	$\varepsilon_{c1,\theta}$	$\varepsilon_{cu1,\theta}$	$f_{c,\theta}/f_{ck}$	$\varepsilon_{c1,\theta}$	$\varepsilon_{cu1,\theta}$
20	1.00	0.0025	0.0200	1.00	0.0025	0.0200
100	1.00	0.0025	0.0225	1.00	0.0040	0.0225
200	0.95	0.0025	0.0250	0.97	0.0055	0.0250
300	0.85	0.0025	0.0275	0.91	0.0070	0.0275
400	0.75	0.0025	0.0300	0.85	0.0100	0.0300

500	0.6	0.0025	0.0325	0.74	0.0150	0.0325
600	0.45	0.0025	0.0350	0.60	0.0250	0.0350
700	0.3	0.0025	0.0375	0.43	0.0250	0.0375
800	0.15	0.0025	0.0400	0.27	0.0250	0.0400
900	0.08	0.0025	0.0425	0.15	0.0250	0.0425
1000	0.04	0.0025	0.0450	0.06	0.0250	0.0450
1100	0.01	0.0025	0.0475	0.02	0.0250	0.0475
1200	0.00	-	-	0.00	-	-

Fig. 2.4 Normalized stress-strain relationship of concrete at elevated temperatures

With regard to the influence of concrete tensile strength to tensile membrane action of reinforced concrete slabs, Foster et al. (2006) reported that at large displacements, concrete tensile strength has a significant contribution to the slab resistance. Therefore, an appropriate constitutive model for concrete tensile strength at elevated temperature is needed.

In comparison with concrete compressive strength, there are limited data on concrete tensile strength at elevated temperatures. EN 1992-1-2 allows the tensile strength to be considered through the reduction factor, $k_{c,t}(\theta) = f_{ct,t}(\theta) / f_{ct}$, given in Eq. (2.6).

$$k_{c,t}(\theta) = \begin{cases} 1.0 & \text{for } 20^{\circ}\text{C} \leq \theta \leq 100^{\circ}\text{C} \\ 1.0 - 1.0(\theta - 100)/500 & \text{for } 100^{\circ}\text{C} < \theta \leq 600^{\circ}\text{C} \end{cases} \quad (2.6)$$

However, if an advanced calculation method is used such as finite element methods, a constituent model is needed. The tensile stress-strain relationship proposed by Youssef and Moftah (2007) is adopted in this research since it can take account of the reduction in the tensile resistance and the bond strength (**Fig. 2.5**).

Fig. 2.5 Tensile stress-strain relationship proposed by Youssef and Moftah (2007)

As can be seen, the uniaxial stress-strain relationship for concrete in tension can be modelled by a linear branch until it reaches the cracking stress, $f_{cr,\theta}$. The modulus of elasticity of the linear branch can be taken to be equal to the initial modulus of elasticity at elevated temperature, $E_{ci,\theta}$. The recommended value for $f_{cr,\theta}$ is $\left(0.6\sqrt{f'_c}\right)f'_c/f'_{c,\theta}$ for flexural tension, where f'_c is the concrete compressive strength at ambient temperature and $f'_{c,\theta}$ is the concrete compressive strength at elevated temperature θ . After cracking, the model of Collins and Mitchell (Collins and Mitchell 1987) was modified by taking account of the reduction in the tensile resistance and the bond strength. The equation for the descending part is given by Eq. (2.7).

$$f_{t,\theta} = \frac{\alpha_1 \alpha_2 f_{cr,\theta}}{1 + \sqrt{500 \varepsilon_{c,\theta}}} \cdot \frac{\tau_{uT}}{\tau_{uo}} \quad \left(\varepsilon_{c,\theta} > \frac{f_{cr,\theta}}{E_{ci,\theta}} \right) \quad (2.7)$$

where α_1 is a factor accounting for the bond characteristics of reinforcing bars, equal to 1.0 or 0.7 for deformed or plain reinforcing bars, respectively; α_2 is a factor accounting for the type of loading; $\varepsilon_{c,\theta}$ is concrete tensile strain at temperature θ .

The bond strength at elevated temperatures, τ_{uT} , is a function of the bond strength at ambient temperature, τ_{uo} , and the temperature of reinforcing bars, T (Xiao and Konig 2004) as follows:

$$\tau_{uT} = \tau_{uo} \left[2.7438 \left(\frac{T}{100} \right)^2 - 3.322 \left(\frac{T}{10} \right) + 105.881 \right] \times 10^{-2} \quad (2.8)$$

It is noted that no recommendation for the limit of concrete tensile strain has been given; thus an appropriate value for numerical models can only be determined by trial and error. A reasonable range for the limit of normal concrete tensile strain would be 0.1% to 0.3%.

2.2.3.2 Modulus of elasticity

A great scatter in experimental results for the initial modulus of elasticity has been observed by a number of researchers (Schneider 1986b; Xiao and Konig 2004; Li and Purkiss 2005). After comparing different models, Youssef and Moftah (2007) concluded that all models predicted $E_{ci,\theta}$ with an acceptable accuracy. The Li and Purkiss model (Li and Purkiss 2005) is adopted in this research because of its simplicity and efficiency. The expression is as follows:

$$E_{ci,\theta} = \frac{800 - \theta}{740} E_{ci} \leq E_{ci} \quad (2.9)$$

where E_{ci} is the initial modulus of elasticity at ambient temperature.

2.2.3.3 Thermal expansion

The main factor affecting the thermal expansion coefficient is the type of aggregate in which the coarse aggregate fraction plays a dominant role (Schneider 1986a). The thermal expansion-temperature curve of the concrete commonly follows that of the aggregate. In the case of weak principal aggregates such as perlite and vermiculite, the thermal strain curve follows that of the paste since the aggregate strength is not sufficient to resist the shrinkage of cement paste.

EN 1992-1-2 gives the values of thermal strain for both normal and light weight concrete. Depending on the aggregate type, the thermal strain of concrete $\varepsilon_{c,\theta}$ can be

determined as follows.

For siliceous aggregates:

$$\varepsilon_{c,\theta} = \begin{cases} -1.8 \times 10^{-4} + 9 \times 10^{-6} \theta + 2.3 \times 10^{-11} \theta^3 & \text{for } 20^\circ\text{C} \leq \theta \leq 700^\circ\text{C} \\ 14 \times 10^{-3} & \text{for } 700^\circ\text{C} < \theta \leq 1200^\circ\text{C} \end{cases} \quad (2.10)$$

For calcareous aggregates:

$$\varepsilon_{c,\theta} = \begin{cases} -1.2 \times 10^{-4} + 6 \times 10^{-6} \theta + 1.4 \times 10^{-11} \theta^3 & \text{for } 20^\circ\text{C} \leq \theta \leq 805^\circ\text{C} \\ 12 \times 10^{-3} & \text{for } 805^\circ\text{C} < \theta \leq 1200^\circ\text{C} \end{cases} \quad (2.11)$$

where θ is the concrete temperature.

2.2.3.4 Spalling

Spalling is the separation of concrete from the surface in concrete structural members when they are exposed to high temperatures. This phenomenon can seriously affect fire resistance and stability of the structure and the occurrences are unpredictable. However, spalling can be minimized in some ways such as reducing the moisture content, reducing the compressive stress, providing additional reinforcement, or using lightweight concrete.

2.2.4 Reinforcing Steel at Elevated Temperatures

EN 1994-1-2 (2005d) covers both hot-rolled and cold-worked reinforcing steel. The thermal and mechanical properties of hot-rolled reinforcing steel are assumed to be the same as hot-rolled structural steel, covered in **Section 2.2.1**, except that the reduction factor is different from that shown in **Table 2.4**.

Table 2.4 Reduction factors for stress-strain relationships of cold worked reinforcing steel at elevated temperature

Steel temperature θ_s ($^\circ\text{C}$)	Reduction factor of tangent modulus $k_{E,\theta} = E_{s,\theta}/E_s$	Reduction factor for proportional limit $k_{p,\theta} = f_{sp,\theta}/f_{sy}$	Reduction factor for effective yield strength $k_{y,\theta} = f_{sy,\theta}/f_{sy}$
20	1.00	1.00	1.00
100	1.00	0.96	1.00
200	0.87	0.92	1.00

300	0.72	0.81	1.00
400	0.56	0.63	0.94
500	0.40	0.44	0.67
600	0.24	0.26	0.40
700	0.08	0.08	0.12
800	0.06	0.06	0.11
900	0.05	0.05	0.08
1000	0.03	0.03	0.05
1100	0.02	0.02	0.03
1200	0	0	0

2.3 Behaviour of Slabs in Fire

From real fires and fire tests at Building Research Establishment (BRE) in Cardington, it was observed that unprotected secondary steel beams lost significant strength and stiffness, and the steel decking debonded from the lightly reinforced concrete (LRC) slab (Bailey and Moore 2000a; Newman *et al.* 2006). The principal load-carrying structural component under fire conditions was the LRC slab. In design at ambient temperature, composite slabs with steel decking are assumed to act as one-way spanning slabs, resisting loads through bending and shear. At elevated temperatures, the unprotected composite slabs undergo very large vertical deflections due to thermal bowing and loss of strength of the steel decking. Thus, the contribution of the steel decking to the load-carrying capacity of composite slabs can be ignored in fire conditions. Furthermore, if the interior secondary beams are unprotected, they also lose significant strength and stiffness. Consequently, the slabs behave as two-way spanning slabs supported by protected edge beams. For these reasons, the behaviour of composite slabs with unprotected interior beams in fire is directly related to the behaviour of two-way reinforced concrete slabs at elevated temperatures.

It was originally thought that the yield line theory developed by Johansen could predict the ultimate strength of reinforced concrete slabs (Hognestad 1953). Yield line theory is a plastic theory used for postulating a failure mechanism of a slab, and parameters of this failure mechanism can be obtained using the principle of virtual

work and the equilibrium equations. This theory employs a flexural failure mode which neglects the in-plane forces and assumes sufficient shear strength.

A number of experiments on laterally restrained reinforced concrete slabs highlighted the reserve load-capacity of slabs above the capacity predicted by Johansen's yield line theory (Ockleston 1955; Park 1964a; Sawczuk and Winnicki 1965). This load-capacity enhancement is due to mobilising membrane actions at large deformations, through tensile membrane action (TMA). Of course, utilising membrane actions implicitly violates serviceability criteria which are not allowed in normal serviceability design. However, for accidental limit states, i.e. fire conditions or progressive collapse, this additional capacity is an inherent part of the resistance and should be made quantifiable if needed. Depending on external supports, the complete load-deflection curve of a two-way reinforced concrete slab (without edge and interior beams) at ambient temperature is shown in **Fig. 2.6**.

Fig. 2.6 Load-deflection curve of a two-way reinforced concrete slab

The behaviour of a two-way reinforced concrete slab *with laterally restrained edges* can be divided into a few phases. As the load increases from point O to point B, i.e. the peak capacity (**Fig. 2.6**), it is postulated that the yield line pattern develops until the collapse mechanism is formed at point B. Concurrently, compressive membrane forces are generated by rigid supports or arching of slab segments between opposite supports. Once the collapse mechanism forms, the slab loses its stability (due to

material or geometric) which usually leads to a transition from the compression zone to the tension zone. Thick slabs normally experience material instability caused by concrete crushing, while thin slabs undergo geometric instability. If sufficient reinforcement is adequately anchored to the supports, the slab can resist the applied loads by membrane forces which begin to change from compression to tension in the central region of the heated slab. Based on the experimental results, Park and Gamble (1980) reported that for slabs with rigid boundaries, the central deflection of the slab at point C is approximately equal to the slab thickness.

From point C to D, the load is initially supported by both TMA and bending until the large stretch of the slab surface causes full-depth concrete cracking, especially over the central region of the slab. As the slab deflection increases, orthogonal reinforcement forms a steel net that supports additional load through pure tensile membrane action. Eventually, increasing deflection leads to fracture of the reinforcing rebars.

The behaviour of a two-way slab *with laterally unrestrained edges* is somewhat different. Tensile membrane action can still develop in the central region of the slab without forming compressive membrane action since there is no lateral restraint. The outer regions of the slab can act as a supporting compressive ring beam to provide the equilibrating forces (**Fig. 2.7**).

Fig. 2.7 Tensile membrane action of horizontally unrestrained slabs (Wang 2005)

A critical condition for tensile membrane action to be mobilised is that the vertical

supports at the slab edges can be maintained and have sufficient strength. The vertical supports for composite slabs refer to the protected edge beams along the edges of the slab panel. The membrane behaviour of composite slab-beam floor systems under fire conditions are discussed in **Section 2.4**.

2.4 Membrane Behaviour of Composite slab-beam Systems in Fire

Tensile membrane action can develop at large deflection provided that the vertical supports at the slab edges can be maintained. In practice, the necessary vertical supports are ensured by protecting the edge beams so that the beam temperature is limited to no more than 620°C at the required fire resistance time. Therefore, the integrity of the vertical supports becomes crucial as the slab and the supporting beams deflect, and large deflections of the edge beams may actually lead to collapse of the beam-slab system.

Fig. 2.8 Failure of floor system subjected to a uniform distributed load. (a) Floor system subjected to UDL; (b) Beam-slab panel failure mechanism; (c) Slab panel failure (Bailey 2004)

For example, consider a beam-slab floor system, as shown in **Fig. 2.8(a)**, which is subjected to a uniform distributed load (UDL). There are two possible failure modes (Bailey 2004). As the load increases, if plastic hinges form in the edge beams (**Fig. 2.8(b)**), a mechanism comprising yield lines across the whole floor plate will occur, in which tensile membrane action cannot be mobilised since there is no restraint against lateral movement. If the beams within the floor plate are designed such that no plastic hinges can form in the edge beams, tensile membrane action can occur in the slab provided that each panel is vertically supported around its perimeter (**Fig. 2.8(c)**). If plastic hinges are formed after membrane action is mobilised, the mechanism shown in **Fig. 2.8(c)** will change to the mechanism shown in **Fig. 2.8(b)**. In the case that the connected columns have insufficient resistance to resist the resulting horizontal forces developed in the beams, catenary action of the beams cannot occur. As a result, the structure will collapse.

An initial review of the behaviour of composite slab-and-beam floor systems shows that the development of TMA is sensitive to the behaviour of edge beams. However, very few experiments have been conducted to study the behaviour of composite slab-beam systems in fire. This gives the impetus for this study.

2.5 Previous Studies on Tensile Membrane Action of Composite Slabs

Six full-scale tests conducted at Cardington by BRE

1. Tests on isolated slab panels
2. Thermal/structural nonlinear FE programs
3. Analytical models
4. Design guide in the UK

1. New interest on testing of the floor assemblies in fire;
2. Design guides outside the UK

Fig. 2.9 Brief history of studies on composite slabs / floor assemblies in fire

Studies on tensile membrane behaviour of composite slabs in fire can be divided into three stages as shown in **Fig. 2.9**. It started with six full scale fire tests conducted in Cardington by British Steel and the Building Research Establishment (BRE). A number of studies have then been conducted, mainly from 2000 to 2007, including experimental, numerical, analytical, and design-related research investigations. A design guidance document, the SCI publication P288 (Newman *et al.* 2006), for steel-framed buildings using composite construction was subsequently developed and applied in the UK. This guide enables structural engineers to take advantage of tensile membrane action to minimize the fire protection to interior secondary beams, and to optimize the cost for fire protection. Although previous studies have been very valuable in developing a greater understanding of structural behaviour in fire, most of the studies from 2000 to 2007 only focused on isolated slab behaviour, rather than on the behaviour of connected floor assemblies. Recognizing this shortcoming, there has been a recent interest from 2008 to the present in membrane behaviour of integrated floor assemblies.

The following sections discuss the relevant previous studies in detail to find out any remaining technical gaps in this area.

2.5.1 Experimental works

Cardington fire tests

In the early 1990s, two fire incidents widely known as the Basingstoke and the Broadgate Fires occurred (Newman *et al.* 2006). Against expectations, these two modern composite steel-framed buildings survived the fire. Thus, the incidents clearly demonstrated that such buildings have superior fire performance which had not been considered in the building codes. However, without test data, the experience of the Basingstoke and Broadgate accidents could not be directly applied in fire safety design of steel framed buildings.

In 1996 a series of fire tests was conducted on a full-scale eight-storey composite steel framed building at Cardington under the leadership of *Building Research Establishment* (BRE). The tests involved different compartment sizes and test

configurations at different locations in the building. The underside of the trapezoidal composite deck and all the steel beams (apart from the perimeter edge steel beams) were unprotected, while the columns were fully protected. Although the maximum compartment temperature in the tests was around 1000°C , the composite floors experienced very large deflections without any structural collapse (**Fig. 2.10**). This was the first time it was observed that a considerable increase in the load-bearing capacity of composite slabs is caused by membrane action.

Fig. 2.10 Large deflections of the composite slabs following the fire tests at Cardington (Newman *et al.* 2006)

Studies on isolated slab panels

Based on the observations from the Cardington tests, to show the existence of membrane action, Bailey *et al.* (2000) conducted an independent test where a 9.5m x 6.5m composite slab was tested to failure at ambient temperature. To ensure nominal vertical displacement around the perimeter, additional columns provided vertical support to the mid-span of edge beams whilst allowing horizontal movement. It was observed that tensile membrane action was mobilised, with the failure load approximately double that predicted by the yield line theory. Based on the test results, Bailey proposed a method to predict the load-carrying capacity of composite slabs, namely, the Bailey-BRE method.

Foster et al. (2004) tested 15 small-scale horizontally unrestrained slabs at ambient temperature to investigate the influence of isotropic and orthotropic reinforcement, together with reinforcement bond strength, on the degree of mobilisation of tensile membrane action. The tests showed clearly that the bond strength between the reinforcement and the concrete plays an important role in the behaviour of the slab in membrane action range. Therefore, the authors proposed that the bond strength should be incorporated into the estimation of the load capacity of the slab in fire.

However, all these tests were conducted at ambient temperature, which may not reflect the actual behaviour of composite slabs in fire. Thus, the valid question was whether the failure mechanism assumed in the Bailey-BRE method took place under fire conditions. Therefore, Foster et al. (2005) conducted additional small-scale tests on horizontally unrestrained slabs at elevated temperatures to investigate the influence of thermal curvature on the failure mechanisms of rectangular slabs. They noticed that the collapse mechanism differs in some respects from that observed at ambient temperature. At ambient temperature, yield line cracks are formed first at low deflection, and then membrane action is mobilised at high deflection. At elevated temperatures, the slab deflects into synclastic curvature first, forming full-depth cracks across the shorter span, and may later generate a less distinctive yield line mechanism.

Bailey and Toh (2007b) commented that the different failure modes observed by Foster et al. (2005) in comparison to that assumed in the simplified design method was due to a high applied static load of 2.8 times the yield line capacity, even before the slabs were heated. This would result in unrealistic behaviour and low reinforcement failure temperatures. In order to further investigate the failure modes of composite slabs in fire, they conducted forty-eight horizontally unrestrained reinforced concrete slabs at both ambient and elevated temperatures. The experiments showed the same failure mode as that assumed in the simplified design method.

It is worth noting that both Bailey and Toh (2007b) and Foster et al. (2005) adopted the assumption of continuous vertical restraint at all times during a fire. Therefore, in their tests, the slab boundaries were vertically restrained and there were no edge beams. Therefore, more tests should be focused on the composite slab panels with actual perimeter steel beams in fire. In such a situation, the in-plane and out-plane deflections of the protected edge beams may adversely affect the ultimate load-carrying capacity of the slabs.

Studies on composite slab-beam floor systems

Recognizing the shortcoming of testing isolated slabs without beams, there has been a recent interest in testing full/small scale integrated floor assemblies under fire conditions. Researchers have had the opportunity for more validation case studies with full-scale fire tests on parts of composite steel-framed buildings.

The first was *Fire Resistance Assessment of Partially Protected Composite Floors* (FRACOF) test conducted in France in 2008. A single composite floor slab panel with two unprotected secondary beams, representative of a corner compartment, was tested under exposure to a 120-minute ISO834 standard fire (**Fig. 2.11**). The main objective of this project is to assess the fire resistance of partially protected composite floors using the design concept in the SCI P288 Publication (Newman *et al.* 2006).

Fig. 2.11 Fracof fire test (Zhao *et al.* 2008)

In the FRACOF test, the slab did not collapse even though the interior secondary beams were unprotected, but the slab failed in terms of integrity. This test is described in detail in **Section 3.4.3** since its results are used to validate the numerical model. A design guide was published and the engineering background with test details was provided (Simms and Zhao 2009). Although providing a very good understanding of the behaviour of composite slab-beam systems, this test did not allow the protected edge beams and the columns to be entirely enclosed within the furnace. Consequently, these members did not attain appreciable temperature or deformation. There was little attention on the effect of edge beam deflections on TMA.

Fig. 2.12 Mokrsko fire test (Ward and Kallerová 2011)

The second full-scale fire test was the Mokrsko fire test in the Czech Republic (Ward and Kallerová 2011). A single-storey building with a strong focus to the unprotected castellated composite beams was tested under a natural fire exposure. The structure represented one floor of a composite office building consisting of four bays with a size of 9m x 6m each as shown in **Fig. 2.12(a)**. After about 61 minutes exposed to a natural fire, three quarters of the structure collapsed. This is the only large-scale structural fire test which has resulted in a structural collapse. The

unexpected collapse of the test is at present unexplained, but the cause may be due to poor construction detailing, such as pinned column bases, and unprotected beam-to-column connections (Abu *et al.* 2009). On the other hand, the floor slab was collapsed probably due to buckling of the edge beam in the corner of the slab as shown in **Fig. 2.12(b)**. When the edge beam buckled, the bolted connection of the primary box girder was subjected to torsion, leading to the loss of shear resistance.

The third was the full-scale slab tests at Tongji University in China conducted by Zhang *et al.* (2009). Four steel-concrete composite slabs with a size of 5.23m x 3.72m each were tested under ISO834 standard fire. Based on the test observations, the authors developed a new method to estimate the load capacity of reinforced concrete slabs under fire conditions taking account of tensile membrane action. In these tests, cracks were formed along the long edges of the slabs (**Fig. 2.13**). However, this crack pattern may be due to the use of very stiff boundary conditions along the slab panels. Therefore, the test may not reflect the actual behaviour of beam-slab systems.

Fig. 2.13 Crack pattern in Guo-Qiang Li's fire tests (Zhang *et al.* 2009)

In 2011, the federal research project “Utilisation of membrane action for the design of composite slab-beam-systems in fire” was conducted by the Technische Universität München in cooperation with the Leibniz Universität Hannover (Stadler

et al. 2011). The main objective of this project was to enable the use of membrane action in Germany, and to investigate the influence of unprotected interior supporting beams between two slab panels. Two medium-scale tests were conducted on composite slab-beam systems in fire. The fire tests revealed a new issue that had not been considered in this research field so far. A large crack occurred nearby the intermediate beam only 19 minutes after the fire test started (**Fig. 2.14**) due to the rupture of entire reinforcement at that region. The slab did not collapse but lost its integrity.

Fig. 2.14 Longitudinal cracks above the interior beam in the first test (Stadler *et al.* 2011)

a) Specimen beam-girder assembly

b) Final cracking after cooling of FA-2

Fig. 2.15 Specimen layout and typical crack patterns (Wellman *et al.* 2011)

Wellman *et al.* (2011) conducted a series of small-scale tests on composite floor assemblies under fire loading (**Fig. 2.15**). These tests were conducted as part of a study intended to consider the elimination of fire protection to interior secondary composite beams in the USA. Partial composite action was achieved in both the main and the secondary beams by using headed shear studs. The observed failure mechanism of the tested specimens included ‘runaway’ failure of the interior beams, followed by ‘runaway’ failure of the edge beams. The term ‘run away’ refers to the failure of the edge beams which was defined as either excessive deflection, or excessive deflection and deflection rate, according to BS 476 (1987) requirements. The authors found that the removal of fire-protection from the interior beams does not have significant effect on the temperature profiles of protected connections and beams, but causes the floor system to rely even more heavily on the composite slab action to carry the gravity loads. Once the interior beams started undergoing failure, the composite slab could not stabilize them or provide adequate force transfer to prevent failure. Therefore, the conclusion of the study was not to recommend removal of fire protection from the interior beams of lightweight composite slabs in the context of current construction practice in the USA. In this test, a five-loading point system was used to simulate uniform distributed load. Therefore, some local concrete crushing was found under the loading points.

In conclusion, the review shows that there are only a few relevant experimental studies on composite slab-beam systems under fire conditions. The effects of rotational restraint as well as of leaving interior beams unprotected on the membrane behaviour of the floor assemblies have not been fully investigated. Besides, most of the previous tests were carried out based on the assumption of continuous vertical restraint at all times during a fire. There is no information on the effect of protected edge beams on the development of tensile membrane action. Therefore, further experimental investigation is needed.

2.5.2 Analytical studies

A number of research studies on membrane action in reinforced concrete slabs have been conducted in the 1960s and a number of analytical models have been provided.

Notable developments are from Wood (1961), Park (1964a; 1964b) and Hayes (1968b; 1968a).

Park (1964a; 1964b) developed a method from the rigid-plastic theory of Powel (1956) and Wood (1961) to predict the capacity of rectangular slabs with various boundary conditions using a rigid-plastic strip approximation and an empirical peak capacity deflection estimate. He estimated the incipient collapse to be at a deflection of 1/10 of span length.

Hayes (1968b; 1968a) made the significant contribution to the development of tensile membrane action theory. He proposed a general method for simply supported rectangular slabs with orthotropic reinforcement. Recently, Bailey (2001a) has adopted Hayes's method with some modifications and developed a new design method for composite slabs under fire conditions, namely, the Bailey-BRE method or the simplified design method (Bailey and Moore 2000a; 2000b).

The Bailey-BRE method

The Bailey-BRE method is based on the rigid-plastic theory with large change of geometry. In this method, the additional slab capacity provided by tensile membrane action is calculated as an enhancement to the small-deflection yield line capacity. Failure is determined by the formation of full-depth tension cracks across the shorter span of the slab, and the limiting condition is defined by the fracture of the mesh reinforcement in terms of the average membrane strain. A limiting deflection of the slab is then proposed including the mechanical and thermal effects. The method, initially developed for isotropic reinforcement (Bailey and Moore 2000a; 2000b), has been extended to include orthotropic reinforcement (Bailey 2003). The change of in-plane stress distributions and the incidence of compressive failure of concrete have been added (Bailey and Toh 2007a).

The method is based on a number of simplifying assumptions. The slab is assumed to be simply supported irrespective of geometric configurations, and the edges are unrestrained from planar movement. This implies that tensile membrane action is

formed in the central region of the slab and a compressive ring is formed around the perimeter of the slab (**Fig. 2.7**). Whilst this assumption is true for slabs with large openings or on the edge of a building, it may not be true for internal slab panels. The other implicit assumption is that vertical supports along the slab panel boundaries at all times during a fire do not deform and are provided by the protected edge beams. This is essential for the slab to bend in two-way synclastic curvature. However, in reality the deflection of the edge beams may be considerable and thus this assumption may be unrealistic.

Although these aforementioned assumptions are safe and reasonable at first glance, there is a lack of experimental works to prove their validity for composite slab-beam systems. Furthermore, the Bailey-BRE method is independent of stiffness of edge beams, and thus does not provide any information on the effects of edge beams and slab-beam interaction on tensile membrane action. Further investigations on these effects are therefore needed. The method is discussed in greater detail in **Chapter 7**.

Method proposed by Usmani and Cameron (2004)

Usmani and Cameron (2004) proposed an alternative three-step method that analyses the load-carrying capacity of laterally restrained reinforced concrete slabs in fire. This method is based on the deflected shape of a slab at large deformation in fire. The temperature distribution in the slab is assumed to vary only through the slab thickness. The effect of beams is also not considered and the compartment edge beams are assumed to deflect much less than the centre of the slab. An Airy stress function is adopted to obtain the deflected shape of the isotropic flat slab using the assumption of linear material behaviour. Then the principle of virtual work is used to establish the load deflection response. To define a point at which failure is considered to occur, the authors used a limiting value for the mechanical strain in the reinforcement spanning in the short direction at the slab centre.

There are several limitations of this design method (Cameron 2003; Usmani and Cameron 2004). The effect of edge beams is not considered but it is implicitly assumed that reinforcement is sufficiently anchored along the boundaries. More

importantly, this method only considers geometric non-linearity with the assumption of material linearity with temperature. However, in fire conditions the slab will experience large inelastic deformation. In this respect, this method may not be reasonable for determining the ultimate load capacity of composite slab in fire (Zhang and Li 2008). Although the method provides extensive understanding of the most fundamental principles that govern the behaviour of composite structures in fire, the method still needs more validation, either by computational predictions or available experimental tests.

Method proposed by Izzuddin and Elghazouli (2004) and Omer and et al. (2006)

Both of the previous approaches (Bailey 2001a; Usmani and Cameron 2004) cannot take account of some factors such as the strain concentrations in reinforcing bars, the bond strength and the reinforcement stress-strain response. These issues are more important for lightly reinforced concrete slabs where the strains at the crack tip are largely governed by the bond-slip characteristics between the reinforcement and the surrounding concrete.

Omer *et al.* (2006) presented a model for the failure assessment of simply supported lightly reinforced concrete slabs. The slabs are unrestrained in-plane taking account of the stress concentrations at cracked locations. The model is based on two previously developed models for the assessment of beams and axial planar restraint slabs (Izzuddin and Elghazouli 2004; Omer *et al.* 2005). The difference is that in the model for simply supported slabs, the edges of the slab are allowed to move in-plane. Thus, this affects the crack width development for a specific level of deformation.

The model considers two variations as shown in **Fig. 2.16**, corresponding to a full depth crack forming at the centre of the slab or two full depth cracks forming at the intersection of yield lines. This conforms to experimental evidences, where it was observed that full depth cracks were formed in either location (Sawczuk and Winnicki 1965; Bailey and Moore 2000a).

Fig. 2.16 Cracks developed along the short span in laterally unrestrained slabs
(Omer *et al.* 2006)

The interesting feature of this approach is that it takes account of the bond stresses developed between the reinforcing bars and the surrounding concrete, but it still has several limitations. With regard to model validations, there is still a lack of the experimental data of the bond strength at elevated temperatures. More importantly, the effects of vertical supports on tensile membrane action have not been investigated.

Method proposed by Zhang and Li (2010)

Fig. 2.17 Division of slab and coordinates of plates (Li *et al.* 2007)

Li *et al.* (2007) presented an alternative model to determine tensile membrane action. In this model, at the limit state the slab is divided into five parts connected by the yield lines and the ellipse (a centre-elliptic part and four rigid parts around) as shown in **Fig. 2.17**. The curved face of the central part under membrane action is assumed

to be an elliptic parabolic which was observed in the test (Zhang *et al.* 2009). The slab is assumed to be both vertically and horizontally restrained along all boundaries. The ultimate load capacity could be obtained based on the equilibrium equations of forces and bending moments. In this model, to obtain the horizontal boundary forces, the slab has to be divided into many strips to take account of both mechanical deflection and thermal expansion.

Although this method gives a reasonable model to simulate tensile membrane action, the procedure to obtain the boundary forces is very complex. Additionally, the global behaviour of the slab has not been considered since the slab has to be divided into strips (Zhang and Li 2010). In order to improve the model, Zhang and Li (2010) present a new method. The difference is that in the latter method, the equilibrium of forces and bending moments in the slab is considered for each rigid plate rather than for the slab strips. Verification of the integrity of the compressive ring is also added.

Both methods proposed by Zhang and Li (2007 & 2010) rely on a controversial assumption that the supporting beams along the slab perimeter are protected and sufficiently stiff during a fire. It means that vertical supports are maintained at all times during the fire exposure. This implicitly ignores the effect of edge beams on the development of tensile membrane action.

Table 2.5 summarised analytical features of the aforementioned approaches.

Table 2.5 Features of previous analytical studies on TMA of composite slabs

<i>Author</i>	<i>Bailey</i>	<i>Usmani & Cameron</i>	<i>Omer et al.</i>	<i>Guo-Qiang Li</i>	<i>Zhang & Li</i>
<i>Year</i>	<i>2000-2007</i>	<i>2004</i>	<i>2006</i>	<i>2006</i>	<i>2010</i>
Boundary Conditions					
- planar unrestrained slabs	√		√		
- planar restrained slabs		√	√		√
- <i>rotational restraints</i>					
- assumption of rigid vertical supports	√	√	√	√	√
Bond strength			√		
Stress concentration			√		
Failure modes					

- compressive ring	√				√
- a full depth crack forming at the centre of the slab	√	√	√		
- two full depth cracks forming at the intersection of yield lines			√		
- five parts divided by yield lines				√	√
Geometric non-linearity	√	√	√	√	√
Material non-linearity	√		√	√	√
<i>Strength and Stiffness of edge beams</i>					

All the aforementioned models adopt the assumption of rigid vertical supports under fire conditions. This implies that the supporting edge beams need to be accurately protected so that they maintain rigid supports for the slab throughout the fire duration. The effects of strength and stiffness of the edge beams on tensile membrane action are thus ignored. These observations gave impetus to the current research work.

2.5.3 Numerical modelling and investigations on tensile membrane action

Numerical modelling of the behaviour of steel structures under fire conditions has extensively developed over the past 20 years. Initially, these models were developed at different research institutions to provide an inexpensive alternative to the standard fire test. Nowadays, they are widely used as an indispensable tool for researchers to obtain a better understanding of various complex modes of structural behaviour.

Wang (2005) provided a good assessment of some well-known computer programs for the analysis of steel-framed structures in fire. The available codes can be divided into two groups. The first group consists of special programs developed at universities and research organisations such as ADAPTIC, FEAST, SAFIR, *Vulcan*; the second group includes large-scale comprehensive commercial finite element programs such as ABAQUS, and DIANA.

One well-known specialist program is the non-linear finite element software ADAPTIC developed at Imperial College by Izzuddin (1991). This program was initially developed to study the non-linear dynamic behaviour of framed structures at

ambient temperature. This was later extended to elevated temperatures by Song, Izzuddin and Elghazouli. ADAPTIC was validated by using the results of the Cardington fire tests. Another specialist program is FEAST developed at the University of Manchester by Dr T.C.H. Liu. This program consists of shell, solid and contact elements and can also model bolts. The program SAFIR has been developed at the University of Liege by Prof. Franssen. One of the most important features of SAFIR is that it can be used for both thermal and structural analyses at elevated temperatures. Users have to conduct thermal analysis for each part of the structure, and then thermal distribution profiles are used for subsequent structural analysis.

The finite element code *Vulcan* has been developed at the University of Sheffield, UK, and is capable of predicting the behaviour of steel, composite and concrete structures at elevated temperatures. The most recent development work was carried out by Huang *et al.* (1999), who proposed a layered approach to model reinforced concrete floor slabs. In this approach, a concrete slab is divided into a number of layers in the thickness direction and reinforcement is treated as a smeared layer. Huang *et al.* (2000) further extended the capability of *Vulcan* to include orthotropic properties of ribbed composite slabs by using different bending stiffnesses in the two directions of a slab. Later, *Vulcan* was developed to have the capabilities of simulating membrane action of reinforced concrete floor slabs.

In the second group of finite element software, ABAQUS and DIANA are commercially available general finite element programs. While ABAQUS has the capabilities to simulate complex structural behaviour under fire conditions, DIANA can model concrete at ambient and elevated temperatures including discrete cracking analysis.

Using SAFIR and *Vulcan*, a number of numerical investigations on tensile membrane action of concrete floor slabs exposed to fire have been conducted (Lim 2003; Foster *et al.* 2006; Abu *et al.* 2007)

Lim (2003) used SAFIR to investigate the effect of different beam sizes on tensile membrane action of two-way slabs. As the beam size decreased, failure of the slabs occurred earlier with larger slab deflections. If the edge beams are strong and stiff enough to allow the slab to behave as if it were on rigid vertical supports, tensile membrane action would be mobilized. In contrast, tensile membrane action could not be mobilised if one or more supporting beams deflected excessively. Lim recommended that fire protection must be applied to the appropriate supporting beams to avoid high aspect ratios of the slabs in fire. The aspect ratios of two-way slabs should be smaller than 2.5.

Foster *et al.* (2006) used *Vulcan* to conduct parametric studies on the load-carrying capacity of reinforced concrete slabs at elevated temperatures. They found that at large displacements and at a temperature up to 200°C, thermal gradient and thermal expansion are the main factors governing the slab resistance. Above 200°C, concrete tensile strength is the dominant factor.

Abu *et al.* (2007) conducted extensive numerical investigations on tensile membrane action of composite slabs in fire. They compared the *Vulcan* analyses with the Bailey-BRE method to investigate the limitations of the method in terms of geometry, support conditions and composition of slab panels. Their results showed that some assumptions of the Bailey-BRE method, which are discussed in **Section 2.5.2**, need to be re-investigated.

Toh and Bernabe (2008) and Omer (2006) compared the load-carrying capacity between rotationally-restrained slabs and simply-supported slabs. It is observed that the rotationally restrained slabs have better fire resistance and smaller deflections than simply supported slabs.

Upon the review of numerical investigations, it can be concluded that tensile membrane action in composite slabs is sensitive to the duration of fire exposure, restraints around the slab edges as well as structural slab configurations. Besides, very few numerical investigations were conducted on tensile membrane action using

a commercial finite element program. An independent investigation is needed. Therefore, the author uses ABAQUS to conduct numerical investigations.

2.6 Conclusions

After the review of the experimental studies and numerical predictions, the following salient points are made concerning tensile membrane resistance of composite slabs at elevated temperatures:

- According to previous experimental investigations, unprotected interior secondary beams at elevated temperatures are assumed to have no contributions to the slab capacity. Even though this observation is correct in terms of strength, due to significant reduction of steel strength at high temperature, this effect on the system behaviour has not been fully investigated. The presence of the interior beams (which are always present in composite floor systems) may affect the magnitude and distribution of stresses in the mesh reinforcement due to compatibility between the steel beams and the concrete slab. This may lead to different failure modes for the floor assemblies compared to the isolated slab panels. Therefore, in Series I of experiment, the author focused on this issue.
- Most of the previous tests were conducted on reinforced concrete unrestrained slabs. The effect of rotational restraint on tensile membrane action has not been investigated yet.
- Previous experimental studies adopt the assumption of continuous vertical restraint at all times during a fire. This is a controversial assumption. Therefore, the author conducted Series II of experiment to study the effect of stiffness of protected edge beams on tensile membrane action.
- An independent numerical study on tensile membrane action using a commercial finite element program, such as ABAQUS, is necessary. This programme contains a long list of many types of finite elements and constitutive material models in its library. It also has extensive analytical capabilities including thermal analysis.

- Previous analytical approaches to predict the peak load capacity of a slab imply that the supporting beams around the slab perimeter are protected and assumed to be vertically restrained during a fire. However, at elevated temperatures, the edge beams will deform and may experience large deflections. This renders the assumption of immovable vertical supports invalid. A semi-analytical approach is needed to take into account the deflection of edge beams on the load-bearing capacity of beam-slab floor systems.

In order to conduct further investigations, material properties of steel and concrete at elevated temperatures are required. Since Eurocodes already provide thermal and mechanical material properties for elevated temperatures, Eurocode constitutive material models are adopted in this research work. Where there is no available test data, recourse is made to the published literature.

The following chapter introduces the numerical models and preliminary numerical studies conducted prior to the experimental tests.

CHAPTER 3 PRELIMINARY NUMERICAL ANALYSES

3.1 Introduction

This chapter discusses preliminary numerical assessments to study the nonlinear behaviour of composite beam-slab systems in fire. The main objectives are to (1) verify the feasibility of applying the FE method to study TMA; and (2) investigate the key parameters affecting the mobilisation of TMA in the composite beam-slab (CB-S) systems in fire. These studies are required before the conduct of fire tests.

FE models have been developed using ABAQUS (2009) because of its extensive analytical capabilities including thermal analysis, and a robust concrete material model. The models are first validated against published test data at both ambient and elevated temperatures, and then a parametric study is undertaken. The parameters investigated include the strength and stiffness of edge beams, aspect ratio of the slab, and rotational edge restraint.

3.2 Solution Strategy

Structures in fire are generally subjected to large deformations. Accurate simulations of this behaviour usually encounter numerical convergence difficulty. To overcome this, ABAQUS/Explicit has been adopted since it uses consistent large-deformation theory. Furthermore, compared to static analysis, the explicit dynamic solver allows pre-processing and provides an easy solution procedure (Hongxia *et al.* 2008). However, since ABAQUS/Explicit is a dynamic analysis program and static solutions are the interest, structures must be loaded quasi-statically to eliminate any inertia effect. In order to determine whether an acceptable quasi-static solution has been obtained, ABAQUS manual proposes that the kinetic energy of the deformed structure should not exceed a small fraction (typically 5% to 10%) of the internal energy throughout the loading steps (Abaqus/CAE 2009). Therefore, before looking at the results that are ultimately of interest, such as stresses and deformed shapes, it

is necessary to evaluate the two energies independently to determine whether the results are reasonable.

To simulate accurately the behaviour of a structure in a fire, the temperature distributions within heated members are important. In this study, the *sequentially coupled thermal-stress analysis procedure* has been used because the thermal field is a major control for the stress analysis, but the thermal solution does not depend on the stress solution. Using the temperature distributions from thermal analysis or experiments, structural behaviour can be obtained.

The behaviour of slab-beam composite floor systems subjected to TMA in fire is three dimensional in nature. Modelling such behaviour consists of material and geometric nonlinearities, large deformations, and contact phenomena. The contact phenomena refer to the interactions of basic components, i.e. endplates, welds and bolts. Since this study focuses on the member behaviour rather than the connection behaviour, a simplified model excluding the contact phenomena has been used.

3.3 Input Parameters

To obtain accurate analysis, the numerous subroutines within ABAQUS and the required input must be well understood. These subroutines not only create the geometry of the model and output the results, but also define material properties, loading schemes and boundary conditions. Properly used material models and loading schemes are critical to capture the structural response accurately. The material properties of concrete and steel at elevated temperatures have been presented in **Section 2.2**. This section discusses some special aspects of material modelling.

ABAQUS (2009) introduces three concrete models, namely, *concrete smeared cracking*, *cracking model for concrete*, and *concrete damaged plasticity*. In the *concrete smeared cracking model*, the presence of cracks is taken into account by the way in which cracks affect the stress and material stiffness at the integration points. The *cracking model for concrete* is most accurate in problems where brittle

behaviour dominates such that the assumption that the material is linear elastic in compression is adequate. However, most importantly, the first two models can only be used in ABAQUS/Standard, and therefore they have numerical convergence difficulty in modelling composite slabs at large deflections.

Therefore, the *concrete damaged plasticity model* in ABAQUS/Explicit has been used. This concrete model is fairly robust which specifies the bi-axial and uni-axial behaviour of concrete subjected to monotonic loading through a variety of input parameters. It is a continuum, plasticity-based, damaged model for concrete, which assumes that the two main failure mechanisms of the concrete material are tensile cracking and compressive crushing (Abaqus/CAE 2009). The yield surface is controlled by two hardening variables, namely, tensile and compressive equivalent plastic strains. **Fig. 3.1** shows the model behaviour in tension and compression.

Fig. 3.1 Concrete plasticity model (Abaqus/CAE 2009)

For compression hardening, ABAQUS suggests that the maximum concrete strains should be calibrated to a particular case. Therefore, the compressive inelastic strains for concrete in compression, the strain $\epsilon_{cu,\theta}$ (corresponding to the maximum compressive strength $f_{c,\theta}$) and the maximum concrete strain $\epsilon_{ce,\theta}$ were taken from EN 1994-1-2 (2005d). For tension stiffening, to avoid potential numerical problems, ABAQUS enforces a lower limit on the post-failure stress equal to one hundredth of

the initial failure stress: $\sigma_t \geq \sigma_{t0}/100$. Thus, if the tensile concrete stress is approximately equal to this limit, the concrete can be considered as failed.

In ABAQUS, steel rebars in reinforced concrete structures can be modelled through two methods: *truss bar elements* that can be defined singly or embedded in oriented surfaces, or *rebar layers*. Modelling reinforcement with *truss bar elements* is normally used in 3D solid elements, while *rebar layer technique* is adopted in 2D models using shell elements. Metal plasticity models are typically used to simulate the behaviour of the rebar material. The effects associated with the rebar/concrete interface, such as bond slip and dowel action, are modelled approximately by introducing ‘tension stiffening’ into the concrete modelling to simulate load transfer across cracks through the rebar.

Material test data are normally in the form of engineering stress-strain curves. In ABAQUS, the steel material model has to be defined by true stress-strain relationships. This allows simulating the structural behaviour more realistically in FE analyses. Therefore, it is necessary to convert material data from engineering stress-strain curves to true stress-strain relationships, especially for large deformation problems where strain localisation in reinforcing bars plays a significant role in the behaviour of reinforced concrete structures. The following equations can be used for the conversion (Abaqus/CAE 2009):

$$\sigma = S(1 + e) \quad (3.1)$$

$$\varepsilon = \ln(1 + e) \quad (3.2)$$

where e is engineering strain; S is engineering stress.

For steel materials, the stress-hardening part can be taken into account, in which the strain parameters are assumed to be the same values recommended in EN 1994-1-2.

Fig. 3.2 shows the normalised true stress-strain-temperature relationships of hot-rolled steel, in which fracture points are indicated. The limiting strain for yield strength in the fire situation, $\varepsilon_{au,\theta}$, was taken as 0.15, and the ultimate strain in the fire situation, $\varepsilon_{ae,\theta}$, as 0.2. The corresponding true strains are 0.140 and 0.182.

Fig. 3.2 Normalised stress-strain-temperature curve for steel

3.4 Verification of Finite Element Modelling

Although ABAQUS has been extensively validated during their development against test data, the way in which a model is created and its results interpreted is extremely important. An FE model simulating a specific problem should be validated by test results so that its limitations for that problem can be well addressed, before it can be used for parametric studies and further investigations.

Therefore, the model verifications are conducted first to obtain reliable models for analysing the structural and thermal behaviour of the assemblies in fire. The works include: (1) Model verifications at ambient temperature; (2) Model verifications at elevated temperatures; (3) Verification of a slab-beam system model; (4) Verification of a heat transfer model.

3.4.1 Model verification at ambient temperature

3.4.1.1 Introduction

This section presents an FE analysis of a two-way simply supported slab tested at ambient temperature. The purpose is to assess the capability of ABAQUS shell and solid elements in predicting ultimate loads of concrete slabs taking account of membrane action.

Bailey and Toh (2007b) conducted forty-eight horizontally unrestrained two-way spanning reinforced concrete slabs at ambient and elevated temperatures. The test programme comprised four series of slabs, two series at ambient temperature and the other two series at elevated temperatures, with two different types of reinforcement (mild steel and stainless steel). Slab M3 in the M-series tested at ambient temperature was chosen for analysis because its failure mode could represent that of two-way simply-supported slabs at large deformations.

M3 with a size of 1.8m x 1.2m x 0.022m was reinforced with mild steel mesh reinforcement (reinforcement ratio of 0.079). The slab was simply-supported with a clear span of 1.7m x 1.1m, giving an aspect ratio of 1.55. At four corners, the slab was lightly clamped so that it could move horizontally with no restraint. **Table 3.1** shows the detailed properties of M3.

Table 3.1 Properties of M3 (Bailey and Toh 2007a)

Thickness	Reinforcement				Spacing		
	Long span		Short span		mm	P	P_{test}
	ϕ (mm)	f_y/f_u (MPa)	ϕ (mm)	f_y/f_u (MPa)		(kN/m ²)	(kN/m ²)
h_s (mm)							
22	1.53	451/487	1.49	454/495	25.4	6.35	12.28

P is the theoretical yield-line load; P_{test} is the maximum test load.

Concrete cube strength f_{cu} measured at 28 days was 35.3MPa, giving a corresponding characteristic cylinder strength f_{ck} of 28.8MPa. According to EN 1992-1-1, the modulus of elasticity could be calculated from Eq. (3.3) giving a value of 32.5GPa.

$$E_{cm} = 22[f_{cm}/10]^{0.3} \quad (3.3)$$

where $f_{cm} = f_{ck} + 8$ (in MPa); f_{cm} is the mean value of concrete cylinder strength;

Two typical failure modes were observed from the M-series slab tests; they were fracture of reinforcement along the short span and compressive concrete failure at

the corners. In M3 test, reinforcement was fractured across the short span as shown in **Fig. 3.3**.

Fig. 3.3 Failure mode of slab M3 (Bailey and Toh 2007b)

3.4.1.2 FE modelling and discussions

In ABAQUS/Explicit, a quarter of the slab was modelled due to symmetric load and geometry. Since the behaviour of two-way spanning concrete slabs is dominated by flexure, both shell and solid elements can be used to model the slab. In order to assess the efficiency of using shell elements compared to solid elements, M3 was analysed using both element types.

Fig. 3.4 shows the FE model using shell elements with a suitable mesh size, element labels, and boundary condition. The mesh consisted of 320 shell elements in a 20x16 grid. Shell element type S4R, a 4-node doubly curved thin or thick shell with reduced integration, was used. The long span (X direction) refers to the longitudinal direction and the short span (Y direction) to the transverse direction. Reinforcement was modelled through smearing of the rebar as a layer within the concrete shell element.

Fig. 3.4 Mesh of M3 using shell,element S4R

Fig. 3.5 Mesh of M3 using solid element C3D8R

Fig. 3.5 shows the FE model using solid element type C3D8R, eight-node linear brick, reduced integration, hourglass control element. The number of elements in the longitudinal and transverse directions was the same as the model using shell elements, but the slab was discretized into four continuum elements through its thickness. Rebars are modelled by truss bar elements embedded in the oriented surfaces of the slab with the same length as the element length. The slab was loaded to the ultimate uniformly distributed load (failure load) at a small incremental rate.

Fig. 3.6 Comparison of the FE simulation with M3 test of Bailey and Toh (2007b)

Fig. 3.6 indicates that both shell and solid elements can predict the slab behaviour very well. However, using solid elements gives a discrepancy at the descending part. It also takes much more computational time and effort compared to using shell elements. Therefore, shell element type S4R was adopted for further investigations.

a) Rebars parallel to the Y direction

b) Rebars parallel to the X direction

Fig. 3.7 Stresses in reinforcing bars across the middle of the slab

The development of stresses in reinforcing rebars parallel to the transverse (Y) and longitudinal (X) directions are plotted in **Fig. 3.7**, to study development of tensile membrane action of M3 as well as its failure mode. No comparison with the test was conducted, because these data were not provided in the test.

Along the X axis, the stresses at elements 1, 5, 10, 15, 20 (their positions shown in **Fig. 3.4**) are plotted. The numbers next to the legends show the distances from the slab mid-span. It is observed that under low load levels, tensile bending stresses are resisted by concrete in tension, thus the stresses in reinforcing bars are very low. After cracking of concrete, between 5kPa and 6kPa, the tensile stresses in the concrete are transferred to the reinforcing steel. It results in a rapid increase of tensile stresses in the reinforcing bars at the central region of the slab (elements 1 and 5). First yield occurs at the mid-span at element 1 under a load of about 6.3kPa. The compressive stress at element 20 is due to the compressive ring formed at the outer edges of the slab. Along the Y axis, a similar development of reinforcement stresses is observed in **Fig. 3.7(b)**. However, the increase of stresses is not as rapid as that in the longitudinal direction because of lower bending moments in this direction.

Although the development of reinforcement stresses cannot be validated by the test results (not reported), it is apparent that M3 developed tensile membrane action by forming tensile stress in the central region and compressive stress at the outer edges of the slab. On the other hand, based on the strain diagram of the rebar layer along the transverse direction with the maximum value of 0.066, M3 seems to fail due to rebar fracture at mid-span in the transverse direction as observed.

3.4.2 Model verification at elevated temperatures

3.4.2.1 Introduction

This section aims to verify material models of concrete and steel in fire and examine the validity of ABAQUS shell element S4R in predicting the fire behaviour of two-way concrete slabs.

A simply-supported slab tested at elevated temperatures from Bailey and Toh (2007b), i.e. slab MF3, was chosen for verification. The slab had mild steel mesh reinforcement and a clear span of 1.7m x 1.1m, giving an aspect ratio of 1.55. The concrete cube strength was 39.1MPa, and the reinforcement ratio was 0.078. **Table 3.2** lists the geometry and material properties of the slab, together with the values of

the theoretical yield-line load at ambient temperature (P) and the applied test load (P_{test}).

Table 3.2 Properties of MF3 (Bailey and Toh 2007b)

Thickness	Reinforcement				Spacing		
	Long span		Short span		mm	P	P_{test}
	ϕ (mm)	f_y/f_u (MPa)	ϕ (mm)	f_y/f_u (MPa)		(kN/m ²)	(kN/m ²)
h_s (mm)							
19	1.54	336/404	1.54	349/420	25.4	4.07	3.66

This slab was tested under a transient heating state, with a load level of 0.9 defined as P_{test}/P . The applied load was kept constant and the slab was heated from its bottom surface up to failure. The furnace provided a heating rate of 300°C/h, up to a maximum temperature of 1000°C. In the simulation, the recorded temperatures at the heated surface were used as input data, and then the structural behaviour was obtained as time-dependent behaviour.

3.4.2.2 FE modelling and discussions

One quarter of the slab was modelled due to symmetry in load and geometry. The slab was restrained from vertical displacement along the two edges to simulate a simply-supported slab. Different meshes were tried and the mesh shown in **Fig. 3.8** gives the best results among the different meshes.

Fig. 3.8 Mesh of MF3 using shell element S4R

This mesh consisted of 1600 S4R shell elements. Reinforcement was modelled via a smeared layer within the concrete shell element. To simulate transient heating, the FE simulation comprises two steps: (1) step 1: apply mechanical loading; (2) step 2: apply heating.

In the simulation of a structure under fire conditions, temperature distribution within the structure plays an important role and governs the accuracy of results. Therefore, to define the thermal gradient through the slab thickness, predicted temperatures were compared with the recorded temperatures. Variations of the temperatures with time are plotted at the heated surface and the reinforcing mesh as shown in **Fig. 3.9**. The graph shows that the simulation matches well with the recorded temperatures if the thermal gradient is defined as $20^{\circ}\text{C}/\text{mm}$.

Fig. 3.9 Comparison of temperatures from test and simulation

Fig. 3.10 Comparison of deflections against mesh temperature

Fig. 3.10 shows the comparison of mid-span deflection against mesh temperature with test data. It can be seen that at the initial stage, the simulation agrees very well with test results. There is only a slight discrepancy when the mesh temperature reaches about 400°C. This is possibly due to concrete cracks in the test leading to greater deflections compared to the numerical model. This phenomenon could not be simulated. In general, the simulation gives conservative overall predictions of the slab response.

3.4.3 Verification of slab-beam system model

3.4.3.1 Introduction

This section develops a numerical model to predict the fire behaviour of composite slab-beam systems.

The Fracof fire test conducted in France (Zhao *et al.* 2008) was adopted for this validation. A single slab panel with two unprotected secondary beams was tested under the exposure of 120min ISO834 Standard Fire. The test was aimed to assess the fire resistance of a full-scale partially protected floor against the SCI P288 design guide (Newman *et al.* 2006).

The slab, representative of a corner compartment, had the overall dimensions of 6.66m x 8.74m. It consisted of four equally-spaced IPE 300 secondary beams spanning in the longer direction and two IPE 400 primary beams (shown in **Fig. 2.11**). The system was supported by four HEB 260 steel columns, using simple connections. Total depth of the slab was 155mm with COFRAPLUS 60 decking. To simulate continuity across the two sides of the slab, reinforcing steel mesh (7mm diameter bars at 150mm centres, placed at 50mm below the top of slab) was welded to two additional steel beams. All the edge beams and the columns were wrapped in 50-mm Cerablanket protection, while the interior beams and the composite slab were left unprotected. The slab resisted a uniform load of 3.84kN/m² which simulated the imposed load at the fire limit state. More details of the test setup can be found in Zhao *et al.* (2008).

3.4.3.2 FE modelling and discussions

A simplified model was proposed, taking account of steel beams, concrete slab and reinforcing mesh. Assuming that contribution of the steel decking on the slab load-carrying capacity was insignificant, the 126mm thick concrete slab was modelled as a flat slab, representative of the effective depth of the composite slab.

The model considered the slab as an isolated slab panel, supported vertically at column positions. The vertical support along the slab edges was naturally provided by the protected edge beams. Rotational restraints along two adjacent edges were used to simulate the continuity across these two edges. **Fig. 3.11** shows the boundary conditions of the model, in which U3 refers to vertical restraint, and UR2 refers to rotational restraint.

Fig. 3.11 Boundary conditions of the model for Fracof test

Shell element type S4R was used to discretize the beams and the slab. Top flanges of the steel beams and parts of the slab above these beams were tied together using surface-based contact interactions. The overlap between two reference surfaces was avoided by apply an offset between two tied surfaces. Reinforcement was modelled via a smeared layer within the concrete shell elements. To simulate transient heating, the first step was to apply mechanical loading to a specified load ratio at ambient temperature. This was followed by heating in the second step.

A comparison between the input temperatures and the measured temperatures was conducted first. For the slab, the comparison was to define a suitable thermal gradient through the slab thickness, while for the beams the comparison was to check the input data. **Fig. 3.12** indicates that the accuracy of temperature simulations was very good. There is only a small discrepancy between the predicted and test results of the steel mesh temperature as shown in **Fig. 3.12(a)**. During the test, as temperature increased concrete cracks developed leading to heat loss. Therefore, the predicted mesh temperature from FEM is always higher than that from test.

Fig. 3.12 Comparison of temperatures from test and simulation

Fig. 3.13 shows the deformed shape of the model, while **Fig. 3.14** plots the deflection comparisons between the numerical predictions and the test results. As observed, the FE predictions are greater than the test results. This may be due to the slightly greater temperatures at the steel mesh compared with the test results (**Fig. 3.12(a)**).

In general, the FE results follow closely the experimental results. The deformed shape is also consistent with that observed from the test.

Fig. 3.13 Deformed shape

Fig. 3.14 Comparisons of the deflections *

* USB denotes unprotected secondary beam; PSB denotes protected secondary beam; MB denotes main beam.

3.4.4 Verification of heat transfer model

This section presents the thermal analysis models which are capable of predicting temperature distribution across composite slabs and protected steel beams. The models were validated with the Fracof fire test results (Zhao *et al.* 2008). In this test, the slab was exposed to the ISO 834 fire from the bottom surface, while the protected main beam was heated from three sides as shown in **Fig. 3.15**. It should be noted that thermal interaction between the beam top flange and the concrete slab was ignored. This would give conservative results because of the heat sink effect of concrete slab which keeps the top flange of the steel beam relatively cool.

Fig. 3.15 Heated surfaces for slab and beams

The material properties for thermal analysis such as specific heat and conductivity, which were not mentioned in the paper, can be found elsewhere (Palm 1994). Details of these properties are given in **Table 3.3**. The density is assumed to be constant at 2300kg/m^3 for concrete, and 7800kg/m^3 for steel.

Table 3.3 Material properties for thermal analysis (Palm 1994)

Concrete			Steel		
<i>Temperature</i>	<i>Specific Heat</i>	<i>Conductivity</i>	<i>Temperature</i>	<i>Specific Heat</i>	<i>Conductivity</i>
$^{\circ}\text{C}$	$\text{J/kg}^{\circ}\text{C}$	$\text{J/ms}^{\circ}\text{C}$	$^{\circ}\text{C}$	$\text{J/kg}^{\circ}\text{C}$	$\text{J/ms}^{\circ}\text{C}$
0	1210	1.94	0	469	60.0
100	1210	1.37	200	525	51.8
115	1000	1.28	400	611	43.5
200	1000	1.14	500	665	39.4
600	1000	0.94	600	727	35.3
1000	1000	0.66	700	796	31.1
1200	1000	0.52	800	873	27.0
1500	1000	0.31	2000	873	27.0

3.4.4.1 Thermal response of composite slab

In the Fracof test, the slab was 155mm thick and was exposed to the ISO 834 fire from below. Similar to the structural model, in the thermal analysis model, it is assumed that the contribution of the steel decking on thermal response of the slab was not significant. Thus, the 126mm thick concrete slab was modelled as a flat slab, representative of the effective depth of the composite slab.

Temperature distribution within concrete slabs can be determined from thermal analysis of a one-dimensional model. The thermal analysis was performed by discretizing the cross section with DC2D8 element (8-node quadratic heat transfer quadrilateral) as shown in **Fig. 3.16 (a)**. The analysis assumed that temperatures only vary across the thickness as was confirmed by the author's test results presented in **Section 5.3.3**. Reinforcing bars were not included in this model, and was assumed not to affect the temperature distribution in the cross section. Temperatures of

reinforcing bars were assumed to be equal to those of the concrete at the same level in the cross section. The effect of concrete cracking on thermal distribution was also ignored.

Fig. 3.16 Cross section of the slab and temperature distribution across the shell

Fig. 3.16(b) shows the temperature distribution across the slab thickness, while the comparison between the predicted and measured temperatures is shown in **Fig. 3.17**. The temperature-time curves were compared at the exposed and unexposed surfaces and at the mid-depth. As can be seen, a good correlation between the two sets of results is obtained.

Fig. 3.17 Comparison of temperatures from Fracof test and heat transfer model

3.4.4.2 Thermal response of protected steel beams

To conduct thermal analysis for the protected beams, protective material was defined as one independent part attached to the beam section through a *surface-based contact interaction* (**Fig. 3.18**). To define the contact property between the master and slave surfaces, *gap conductance* was adopted and specified as a function of clearance incorporated in the contact definition through contact property assignment. With regard to boundary conditions, ABAQUS allows five types of thermal boundary conditions. They are: *prescribed temperatures at particular points*, *prescribed fluxes* (concentrated fluxes or distributed fluxes), *film conditions*, *radiation conditions* at particular points or on particular surfaces, and *natural boundary condition*. In this model, thermal boundary condition is simulated by defining *prescribed temperatures at particular points* since the measured temperatures are given in a form of temperature-time curves. The beam was discretized by eight-node, quadratic-interpolation, heat-transfer elements DC2D8.

Material properties of concrete and steel for thermal analyses are presented in **Table 3.3**. Properties of the protective material were assumed as follows: density 800kg/m^3 ; specific heat $1700\text{J/kg}^\circ\text{C}$; and conductivity $0.2\text{J/ms}^\circ\text{C}$. This information was not given in the paper (Zhao *et al.* 2008).

Fig. 3.18 Cross section of the beam with fire protection layer

Fig. 3.19 Comparison of predicted and recorded temperatures of the protected main beam

Fig. 3.19 shows the comparison between the predicted and measured temperatures of the protected main beam in the Fracof test. As shown, good agreement between the test and the simulation is obtained. It can be concluded that the proposed thermal analysis models can predict quite well the thermal responses of composite slabs and protected beams.

3.5 Parametric Study

3.5.1 Introduction

The proposed models, viz. structural response and thermal analysis, have shown the capability to simulate the behaviour of composite slab-beam systems in fire. Therefore, these models are used to conduct a parametric study to determine the main parameters for the experimental programme.

Due to laboratory constraint, it was only possible to conduct small-scale specimens. Therefore the dimensions of the specimens were obtained by scaling down to one-fourth of prototype slab panels. The investigated slab panel was an interior slab panel which consisted of secondary and primary beams supporting a concrete slab as shown in **Fig. 3.20**.

Fig. 3.20 Prototype interior slab panel

The slab panel was designed to carry a permanent load of 1.70kN/m^2 and an imposed load of 3.0kN/m^2 (recommended loads for office buildings in BS EN 1991-1-1 (2002)) in accordance with BS EN 1994-1-1 (2004). All steel beams of grade 355 were designed with full composite action. After the beam sizes were obtained, the edge beams were assumed to be fully protected using intumescent paint in order that its temperature was limited to less than 620°C after 60min of heating. The composite slab and the interior beams were left unprotected. Details of the design can be found in **Appendix A**.

Fig. 3.21 Control slab and FE model for parametric study

In the parametric study, the slab obtained by scaling down to $\frac{1}{4}$ of the $9\text{m} \times 9\text{m}$ slab panel was chosen as the control slab which was uniformly loaded with a predefined load level of 0.6 with a value of 19.20kN/m^2 (**Fig. 3.21(a)**). The load level is defined as the test load divided by the yield-line load at ambient temperature ($P_{test}/P_{yield,20}$). After being loaded up to the desirable load level, the slab was heated from its bottom surface.

The slab thickness was 50mm with a cylinder strength of 30MPa. Mesh reinforcement of 3mm diameter was placed at a distance of 25mm from the top surface. The yield strength of reinforcement was 500MPa.

To simulate an interior panel, the slab was extended beyond the edge beams a segment of 0.375m which formed the effective width for the edge beams. For simply-supported beams in buildings, EN 1994-1-1 (2004) specifies an effective width equal to $L/8$ on each side of the steel web, in which L is the beam span. Therefore, the width of 0.375m is sufficient to allow the edge beams to behave as T-flange beams. For the purpose of numerical analyses, it was assumed that the effect of in-plane restraint from surrounding slabs outside the slab outstand on tensile membrane action is negligible. Thus, no axial restraints along the slab edges were applied.

Since the applied load and geometry of the slab are symmetric, only a quarter of the slab-beam system was modelled (**Fig. 3.21(b)**). Steel decking was not modelled in recognition of de-bonding of steel decking under severe fire conditions.

Properties of the edge beams which may affect the behaviour of the floor assemblies include the strength, torsional rigidity GI_t , bending stiffness in major axis EI_y and bending stiffness in minor axis EI_z . Normally, steel-framed composite buildings use I-section steel beams for the main and the secondary beams. I-section steel beams provide little bending stiffness about the minor axis EI_z ; they support load mainly by in-plane bending. Therefore, three remaining factors such as strength, bending stiffness in major axis and torsional rigidity were chosen for the parametric study. Besides, the effects of unprotected interior (secondary) beams and rotational restraint on the fire behaviour of the assemblies were also investigated.

The points of interest are the maximum concrete compressive strain, rebar yielding over the supports or at mid-span, horizontal reactions along the centre lines (which indicate either tension or compression membrane forces in the slab), and deflections of the slab and the beams with rising temperature. These points provide valuable insight into the membrane behaviour of the floor assemblies in fire.

In simulation, 'failure' point was defined according to EN 1363-1 (2012). In terms of load-bearing capacity, for flexural members, EN 1363-1 stipulates the failure

criterion either by limiting deflection or limiting rate of defection (which ever occurs sooner) as shown in Eqs. (3.4) and (3.5). Based on these criteria, fire resistance of the assemblies can be determined. Therefore, the slabs were considered to have failed when its deflection reached 253mm, or its rate of deflection was 11.3mm/min.

$$\text{Limiting deflection: } D = L^2 / (400d) \text{ mm} \quad (3.4)$$

$$\text{Limiting rate of deflection: } \frac{dD}{dt} = \frac{L^2}{9000d} \text{ mm/min} \quad (3.5)$$

Before studying structural behaviour, a series of thermal analyses were needed because under fire conditions, the structural behaviour of members is adversely affected by their temperature profiles.

3.5.2 Heat transfer analyses

Using the validated model in **Section 3.4.4**, thermal analyses were performed for the 50 mm thick concrete slab, the 127x114x29-joists protected main beam (MB), the 76x76x15-joists protected secondary beam (PSB) and the 76x76x15-joists unprotected secondary beam (USB). The slab was exposed to the ISO 834 fire curve from the bottom, while the beams were exposed to the same fire curve from three sides (**Fig. 3.15**). Material properties of concrete and steel for thermal analyses are presented in **Table 3.3**. Material properties of protective materials are assumed as follows: density 800kg/m³; specific heat 1700J/kg°C; and conductivity 0.2J/ms°C. The simulation results are shown in **Fig. 3.22**. Using these thermal data, structural response of the model can be obtained.

Fig. 3.22 Heat transfer analyses for the beams and the slab

3.5.3 Mesh convergence study

This section presents the analysis of the control slab (**Fig. 3.21(a)**) conducted with a different number of shell elements and mesh sizes. The purposes were to investigate the effect of different number of elements and to define whether the mesh selected is adequate for accuracy. A finer mesh typically results in more accurate solutions but the associated computational time is increased as well.

Fig. 3.23 Different meshes used for the analysis

The slab uniformly loaded with a value of 19.20kN/m^2 was analysed with four different grids as shown in **Fig. 3.23**. The first three meshes had the same refinement as for the beams. Mesh 4 has the same mesh as mesh 3 for the slab but a finer mesh for the beams. To ensure node compatibility between the slab and the beams, the top flange of the beams and parts of the slab above these beams were tied together using surface-based contact interactions. An offset between the two tied surfaces was applied to avoid an overlap between the two reference surfaces.

Fig. 3.24 shows the central deflection of the slab against mesh temperature. The graph shows that the four FE meshes predict almost identical response up to a mesh temperature of 600°C . Upon 600°C , the deflection curve with the 25×25 grid (meshes 3 & 4) diverges slightly from that with the 15×15 grid (mesh 2). The corresponding deflections at a mesh temperature of 700°C are 280mm and 250mm for mesh 3 and mesh 2, respectively.

Fig. 3.24 Central slab deflection with different FE meshes

Fig. 3.25 Midspan deflection of the beams with different FE meshes

A comparison of the midspan deflection of the edge beams with different meshes (meshes 3 & 4) is shown in **Fig. 3.25**. It can be observed that the finer mesh, i.e. mesh 4, gives slightly more accurate solution in predicting the behaviour of the beams. However, this mesh cannot be used because when the mesh temperature reaches 250°C, the analysis was terminated due to difficulty in convergence. This is because many elements have distorted excessively with such a small mesh.

Numerical analyses show that the effect of using different meshes becomes apparent when the slab approaches failure. With a finer mesh, the strain in reinforcing bars can be detected more accurately. Globally, mesh 3 (with an element size of about 60mm) gives reasonably accurate solutions for the beams and the slab with an acceptable computational time. Therefore, this mesh was chosen for further analyses.

3.5.4 Effect of unprotected interior beams

Two cases were analysed to study the effect of unprotected interior beams on the slab behaviour. The first case was the control model with two interior beams as shown in **Fig. 3.21(a)**. The second case had similar properties but without any interior beams. **Table 3.4** shows the properties of edge beams for these two cases.

Table 3.4 Case studies for strength of edge beams

Case study	Denote	MB (joists)	SB (joists)	I_y - MB	I_y - SB	Note
1	ST0	127x114x29	76x76x15	I_1	I_2	No interior beams
2	SE1	127x114x29	76x76x15	I_1	I_2	Control model with two interior beams

Fig. 3.26 shows the comparison of deflections of the slabs and the edge beams against time. It is clear that slab ST0 without interior beams experiences significantly greater deflections compared to slab SE1 with two interior beams, even though the trend is similar. The failure times (which correspond to EN 1363-1 criteria) are 8.86min and 115min for ST0 and SE1, respectively. **Fig. 3.26(b)** also indicates that the edge beams in ST0 have greater deflections than those in SE1. It is due to different load paths from the slabs to the edge beams for these two cases. In ST0, load is transferred directly from the slab to the edge beams, while in SE1 load is transferred from the slab via the interior beams to the main edge beams.

Fig. 3.26 Mid-span deflection of slabs and edge beams for ST0 and SE1

It can be observed in **Fig. 3.26** that the presence of unprotected interior beams reduces the slab central deflection; although at elevated temperature the stiffness and strength of interior beams have significantly reduced. This is probably due to compatibility of deflections between the slab and unprotected interior beams. Through the interior beams, which are supported directly by the protected edge beams, applied load can be transferred to the edges in a more direct manner.

3.5.5 Effect of strength of steel beams

Case studies

This section discusses the effect of strength of steel beams on the behaviour of composite slab-beam systems in fire. Since yield strength is the most important factor influencing the beam strength, the yield strength of the main and secondary beams of the control model was chosen for analysis. Three cases as shown in **Table 3.5** were studied, in which model SE1 was the control model.

Table 3.5 Case studies for strength of perimeter beams

<i>Case study</i>	<i>Denote</i>	<i>Yield strength of main beam (MPa)</i>	<i>Yield strength of secondary beam (MPa)</i>
1	SE1	355	355
2	SE2	355	275
3	SE3	275	355

Fig. 3.27 shows the comparison of midspan deflection of the slabs against time. After the loading stage, the slabs have small deflections with the maximum value of 2.6mm although the load level of 0.6 is considered high. Therefore, the deflection at the loading stage is not shown for clarity. At the heating stage, under the exposure of ISO 834, the slab deflection increases rapidly, reaching a peak value of about 300mm after 250min. The maximum value is about $6.0h_s$ (h_s is the slab thickness) when the reinforcement temperature reaches 720°C.

Fig. 3.27 Midspan deflection of the slabs with different steel grades of the beams

As can be seen, three investigated cases give similar deflection-time curves of the slab. Comparing models SE1 and SE3, it is found that the yield strength of the protected main beam has no effect on the slab deflection. This may be due to the temperature of the protected main beam being kept below 350°C (see **Fig. 3.22(a)**), which allows the main beam to maintain its strength in fire.

Comparing models SE1 and SE2, it is found that when the yield strength of the secondary beam decreases, the slab deflection increases. However, the graph shows that the strength of secondary beam has little effect on the slab deflection.

Variation of membrane forces with time

The membrane forces of the control model (SE1) using mesh 3 across the midspan sections are plotted after loading phase, at 60, 120 and 200min after heating in **Fig. 3.28** (see the axes in **Fig. 3.21 (b)**). In this figure, positive and negative values indicate tensile and compressive forces, respectively. The membrane forces across the midspan sections are the reaction forces taken from the nodes along the two axes of symmetry. It is noteworthy that due to symmetry, a quarter of the control slab was modelled.

a) Along X axis (parallel to secondary beams)

b) Along Y axis (parallel to main beams)

Fig. 3.28 Variation of membrane forces in the control slab with time

After the loading phase, the deflection of the slab is very small and the load is mainly resisted by bending action. After 60min of heating, the tension tractions along the X axis start to mobilise. However, due to the presence of the main beam, the width of the tensile region is quite small. Along the Y axis, the edge and interior beams also restrain the development of TMA. Due to composite action between the edge and interior beams and the slabs, the beams act as T-flange beams. As a result, parts of the slab above the beams are still in compression. For example, above the position of the main beam (**Fig. 3.28(a)**), the concrete slab is in compression, and the tensile region is about 500mm after 60min of heating. However, with further heating, the width of compression region reduces and the magnitude of compression

also reduces from 16kN to 8kN. This is due to reductions in strength and stiffness of the mesh reinforcement as temperature increases.

After 120min of heating, as the slab deflection increases, the central tensile region expands quickly. Across the short span (along the X axis), the tensile forces have a uniform distribution. Along the Y axis, the interior beam changes from bending to catenary action. As a result, the tensile forces develop considerably. The corresponding maximum deflection of the slab is 258mm or $5.2h_s$ at a temperature of the interior beam of about 1000°C (**Fig. 3.22(c)**).

After 200min of heating, the load is resisted by pure tensile membrane action since the large stretch of the slab causes full-depth concrete cracking. The in-plane membrane mechanism at failure comprises a compressive ring of concrete around the slab perimeter and tensile membrane action in the central region of the slab. The width of the tensile region is about 750mm with the maximum deflection of $6.1h_s$ (**Fig. 3.28(a)**).

3.5.6 Effect of bending stiffness of protected edge beams

Case studies

This section investigates the effect of bending stiffness of the edge beams on the development of TMA. Three cases are analysed with different beam sizes for one slab configuration (the control model) as shown in **Table 3.6**, in which I_y stands for bending stiffness of the beams in major axis. SE1 is the control model, ST2 has the stiffness of secondary edge beams (I_y - PSB) increased to twice that of SE1. ST3 has the stiffness of main beams (I_y - MB) increased to twice that of SE1. Three slabs have similar properties for the interior beams. For each case the comparison is based on 'so-called' failure time and distribution of membrane forces.

Table 3.6 Case studies for bending stiffness of protected edge beams

Case study	Denote	MB (joists)	PSB (joists)	I_y of MB	I_y of PSB	Note
1	SE1	127x114x29	76x76x15	I_1	I_2	Control model
2	ST2	127x114x29	89x89x19	I_1	$2I_2$	Increase PSB section
3	ST3	152x127x37	76x76x15	$2I_1$	I_2	Increase MB section

Fig. 3.29 shows the comparison of the midspan slab deflection against time. The failure times, which correspond to the slab deflection of 253mm, are 113min, 150min and 175min for SE1, ST2 and ST3, respectively. As can be seen, as the stiffness of the edge beams increases, failure of the slab occurs later with a smaller deflection. Increasing the stiffness of protected secondary edge beams has a more beneficial effect on the slab deflection than increasing the stiffness of main beams. Increasing the stiffness of main beams affects clearly the slab deflection when it has undergone large deflection, beyond 200mm or $4.0h_s$.

Fig. 3.29 Midspan slab deflections with different bending stiffness of edge beams

Variation of membrane forces with time

A comparison of the distribution of membrane forces across the midspan sections at failure (corresponding at the deflection of 253mm or $5.1h_s$) is shown in **Fig. 3.30**. In this figure, positive values indicate tensile forces and negative values compressive

forces; and X and Y directions are shown in **Fig. 3.31**. As can be seen, in ST2 greater tensile membrane forces are mobilized compared to that in SE1. However, ST3 provides less tensile membrane forces at this deflection than SE1 and ST2. This may be explained by the effective width of the T-flange edge beams. As the stiffness of the edge beam increased, the stiffer edge beam can attract greater bending moments which, in turn, lead to greater compressive forces in the concrete slab above the edge beam. It is noteworthy that temperature of the edge beams at failure is only about 300°C. Therefore, estimating the enhancement of the slab load-bearing capacity due to tensile membrane action at elevated temperatures without considering the edge beams may not be accurate.

Fig. 3.30 Distribution of membrane forces at the slab deflection of $5.1h_s$

Fig. 3.31 shows the plastic strain components of reinforcement along the X direction and of the concrete at the slab top surface, and **Table 3.7** summarizes the simulation results of the slabs. PE11 is the plastic strain in reinforcement, and PE12 is the shear strain component of the concrete at the slab top surface. Based on the plastic strains of concrete and mesh reinforcement, the failure modes could be predicted. It can be seen that for all cases the strain of reinforcing bars over the edge beams and the unprotected interior beams is always greater than that at the mid-span. This may be due to local bending in the vicinity of the beams due to compatibility of deflections between the slab and the beams. This local bending causes strain concentrations. Therefore the fracture of reinforcement over the protected edge beams will occur first, and then the slab may fail by fracture of reinforcement over the unprotected

interior beams or by failure of the compressive ring at the corners as shown in **Fig. 3.31**. Experimental results presented in **Chapters 5 and 6** lend support to these conclusions.

a) PE11 of reinforcement along X direction b) PE12 of the concrete at the slab top surface

Fig. 3.31 Predicted failure modes based on the plastic strain components of reinforcement and the concrete at the slab top surface

If interior beams are over-stiff, a different failure mode may occur. As can be seen in **Fig. 3.30(b)** in ST2, part of the slab is still in compression. This indicates that as the stiffness of secondary edge beams increases, it seems to be more difficult to mobilize TMA in both directions, resulting in the possible formation of plastic hinges in the main beams. Tensile membrane action that is supposed to develop in two directions of the slabs becomes catenary action that develops in only one direction. The failure mechanism may change to ‘folding’ mechanism.

The above observations are different from Bailey’s finding (Bailey and Toh 2007b) that the slab will fail by fracture of reinforcement across the shorter span and interior beams have no effect on the mobilisation of TMA because of significant strength and stiffness degradation of steel material under fire conditions. It is because the failure modes of the floor assemblies involve the ‘beam-slab failure mechanism’ with beam-slab interactions, and not solely the slab failure mode. However, these observations are based on numerical investigations. They need to be confirmed by test results. Therefore, an experimental programme was proposed and conducted by the author to provide evidences for these conclusions.

Table 3.7 Summary of FE results at failure (at deflection of $5.1h_s$)

<i>Case</i>	<i>Denote</i>	<i>MB disp.</i>	<i>PSB disp.</i>	<i>Width of tensile region</i>		<i>Max strain in reinforcement over MB</i>	<i>Max strain in reinforcement over PSB</i>	<i>Max strain in reinforcement over USB</i>	<i>Max strain in reinforcement across midspan</i>	<i>Max strain in concrete at corners</i>
				<i>along X axis</i>	<i>along Y axis</i>					
		(mm)	(mm)	(mm)	(mm)	(PE11)	(PE11)	(PE11)	(PE11)	(PE12)
1	SE1	-6.7	-30.4	690	750	0.124	0.1938	0.0241	0.0134	-0.0604
2	ST2	-14.2	-33.2	810	750	0.154	0.1479	0.0192	0.0114	-0.0535
3	ST3	-5.3	-40.1	810	750	0.103	0.1662	0.0204	0.0102	-0.0515

Note: USB denotes unprotected secondary beam; PSB denotes protected secondary beam; MB denotes main beam; PE: plastic strain components.

3.5.7 Effect of torsional rigidity of protected edge beams

As mentioned, torsional rigidity GI_t of protected edge beams may have significant effect on the slab behaviour. Therefore, this section provides a numerical investigation on the torsional rigidity of protected edge beams on its behaviour.

The control model SE1 was chosen as a basis for comparison. Three cases were investigated, namely TB1, TB2, TB3 as shown in **Table 3.8**. In order to increase the torsion constant I_t of the sections but not alter their bending stiffness in the major or the minor axis, I-beam sections were modified to rectangular hollow sections (RHS) with the same width, height, and web thickness. Both protected main beams (MB) and protected secondary beams (PSB) of TB1 were changed to RHS. Their torsional rigidities are increased 68 and 50 times, for MB and PSB, respectively. For TB2, only the MB section was modified, while for TB3 only the PSB section was changed.

Table 3.8 Case studies for torsional rigidity of protected edge beams

Case study	Denote	MB	PSB	I_y, I_t of MB	I_y, I_t of PSB	Note
1	SE1	Joists 127x114x29	Joists 76x76x15	I_1, I_{t1}	I_2, I_{t2}	Control model
2	TB1	RHS 127x114x29	RHS 89x89x19	$I_1, 68I_{t1}$	$I_2, 50I_{t2}$	Change MB & PSB to RHS
3	TB2	RHS 152x127x37	Joists 76x76x15	$I_1, 68I_{t1}$	I_2, I_{t2}	Only change MB to RHS
4	TB3	Joists 152x127x37	RHS 76x76x15	I_1, I_{t1}	$I_2, 50I_{t2}$	Only change PSB to RHS

Fig. 3.32 shows the comparison of the midspan slab deflections for four cases against time. Based on the failure criteria in EN 1363-1 (Eqs. (3.4) and (3.5)), SE1 fails after 113min of heating, while TB3 fails after 140min of heating. TB1 and TB2 have not failed yet even after 200min of heating.

Generally, increasing the torsional constant of the protected edge beams, either the main or the secondary beams, reduces the slab deflection and failure occurs later.

Torsional constant of MB is found to have a greater effect on the slab behaviour than that of PSB. This is because the main beam is the support for the interior beams. As the torsional constant of MB increases, the rotation at the interior beam-to-main beam connections reduces, resulting in a smaller slab deflection.

Fig. 3.32 Midspan slab deflections with different torsional rigidity of edge beams

After 113min of heating (failure of SE1), the slab deflection is 253mm, 208mm and 238mm, for SE1, TB2 and TB3, respectively. The torsional rigidity of PSB is increased 50 times, but the deflection only reduces 6%. Additionally, the torsional rigidity of MB is increased 68 times, but the deflection only reduces 18%. Therefore, it can be concluded that the effect of torsional rigidity of protected edge beams on the slab behaviour is not significant.

3.6 Conclusions

Preliminary numerical analyses were conducted to study the key parameters affecting the behaviour of composite slab-beam systems in fire. The numerical simulations employed an explicit dynamic solver in ABAQUS to overcome convergence difficulty. The proposed models, *viz.* heat transfer and thermal/structural models, were validated against published test data at both ambient and elevated temperatures. It is demonstrated that the FE models give reasonable accuracy compared to test results, providing an efficient, economical and yet accurate tool to study tensile membrane behaviour of the floor assemblies in fire.

Using the validated models, a parametric study was subsequently conducted. The following conclusions can be drawn:

- (1) The presence of unprotected interior beams reduces the slab deflection, though at elevated temperatures the stiffness and strength of the interior beams have significantly reduced. This is because with the presence of the interior beams helps to stabilise the floor system, therefore the load from the slab transferred to the edge beams in a more smooth way.
- (2) Steel grade of protected edge beams has little effect on the behaviour of composite slab-beam systems. This may be because in this parametric study, the temperatures of the edge beams are below 400°C at which the beam strength still has not been reduced by the effect of elevated temperatures.
- (3) As the stiffness of the protected edge beams increases, failure of the slab occurs later with a smaller deflection. On the other hand, increasing the stiffness of protected secondary edge beams has a more beneficial effect on the slab deflection than increasing that of main beams.
- (4) Increasing torsional rigidity of the protected edge beams, either main or secondary beams, has a positive effect on the slab behaviour. However, this effect is not significant.

Based on the numerical investigations, it was found that the parameters which play a significant role in behaviour of the floor assemblies under fire conditions are (1) the presence of interior beams, and (2) bending stiffness of the protected edge beams. Therefore, a testing programme has been proposed to study these effects experimentally. **Chapter 4** presents design of specimens, test setup in detail and preparatory works conducted.

CHAPTER 4 EXPERIMENTAL PREPARATION

4.1 Introduction

The numerical analyses presented in **Chapter 3** show that the presence of interior beams, rotational restraint, and stiffness of the edge beams play a significant role in tensile membrane behaviour of composite substructures in fire. Although providing some insight into the behaviour of the composite slab-beam system, the numerical investigations have yet to be verified by tests. Furthermore, the limitations of the numerical studies include: (1) failure modes of the CB-S systems could not be clearly identified; (2) local failures such as concrete crushing and fracture of reinforcing bars cannot be performed. Therefore an experimental programme is needed to provide more information of the failure mechanisms of such structures. For this purpose, two series of composite beam-slab systems were designed and tested to failure under fire conditions in Nanyang Technological University in 2011 and 2012.

This chapter describes the preparatory work including the setup of a new electric furnace, design of specimens and test setup.

4.2 Setup of New Electric Furnace and Fire Exposure

In order to obtain the objectives of the experimental programme, a new electrical heating furnace was designed and fabricated. Due to laboratory constraint, the inner dimensions of the furnace were limited to 3.0m long, 3.0m wide and 0.75m high as shown in **Fig. 4.1**. It consisted of two individual heating panels with a maximum operating temperature of 1000°C. The furnace height of 0.75m allowed the specimens to undergo large deflection with no limitation, because the preliminary numerical analyses in **Chapter 3** showed that the maximum deflection of the slabs was about 300mm. Four openings at the furnace bottom surface allowed the specimens to be connected to the column bases. The furnace was firmly secured onto a strong stainless steel housing with great structural rigidity.

a) Overview

b) Inside view

Fig. 4.1 Electric heating furnace

In standard fire tests, specimens are commonly subjected to ISO 834 fire curve, in which temperature rises rapidly in the initial phase of heating. Due to limitations of the power supply (only 500Amps), the furnace could not simulate the ISO 834 standard fire curve. This posed little problem as the main aim of the furnace was to achieve the maximum temperature at the mesh reinforcement in the slab. The specimens could not be used to obtain fire resistance as they were one-quarter scale models. Heat transfer across small scale specimens will not be realistic when compared to full scale specimens. On the other hand, the greatest concern here is the structural behaviour of the beam-slab system rather than its fire resistance.

A trial test without a specimen was conducted to determine the furnace heating rate. Seven thermocouples were used, with five measuring the temperature inside the furnace and two measuring the interior air temperature. **Fig. 4.2** indicates that the furnace temperature could reach 1000°C within 58min, giving a heating rate of about 17°C/min. This heating rate is within the allowable range (5°C/min to 20°C/min) specified by BS 5950-8 (2003) as a practical range for steel sections. Therefore, the fire curve was set as follows: temperatures were increased from room temperature up to 1000°C within 60min, and then kept constant during two hours after that. This was because the slabs were estimated to have failed within two hours under fire conditions.

Fig. 4.2 Heating rate from trial test without a specimen

4.3 Description of Test Specimens

4.3.1 Design of specimens

Fig. 4.3 shows the prototype building and the test model. The investigated zone was an interior slab panel located at the middle of the floor. This prototype floor was designed in accordance with EN 1993-1-1 (2005a) and EN 1994-1-1 (2004c) for gravity loading case to carry an imposed load of 3kN/m^2 , inclusive of floor finishes and ceiling loads of 1.7kN/m^2 .

The prototype composite slab was designed with a re-entrant composite floor deck ComFlor 51 (Corus). As discussed in **Section 2.3**, the contribution of steel decking to the load-bearing capacity of composite slabs may be ignored in severe fire conditions. However, in this experiment, to protect the heating elements from concrete spalling the slabs were cast onto a 2mm thick steel sheet with small pre-drilled holes.

Design information of the prototype slab panel and the control slab is listed in **Table 4.1**. Detailed calculations for the prototype building including composite slab, secondary and primary beams can be found in **Appendix A**.

Fig. 4.3 Plan view of the prototype building

Table 4.1 Design information of the prototype slab panel and the control slab

	Prototype slab panel	Control specimen
Slab geometry		
Length, L_y	9.0m	2.25m
Width, L_x	9.0m	2.25m
Slab thickness, h_s	150mm (overall slab depth)	55mm (concrete slab)
Primary beam	UB 533x210x82	W130x130x28.1
Secondary beam	UB 356x171x45	Fabricated section 80x80x17.3
Interior	UB 356x171x45	Fabricated section 80x80x17.3
Concrete properties		
Cylinder strength, f_{ck}	30MPa	30MPa
Structural steel properties		
Yield strength, f_{yk}	S355	S355
Reinforcing steel properties		
Reinforcement ratio	0.2%	0.16%
Reinforcing mesh	$\phi 8a150$	$\phi 3a80$
Yield strength, f_{yk}	500MPa	500MPa

* *Fabricated section 80x80x17.3 means the section is 80mm high, 80mm wide and 17.3kg/m mass.*

Due to laboratory constraint, the principal dimensions of test specimens, 2.25m x 2.25m, were obtained by scaling down by *one quarter* from the prototype slab panel, 9.0m x 9.0m. The sizes of the beams and the slab were scaled, but the key factors which could adversely affect the membrane behaviour were kept constant as those of the prototype panel, such as the reinforcement ratio of the slab, ratio of bending stiffness about the major axis EI_y of the beams relative to that of the slab.

One of the main purposes of current research was to investigate the behaviour of CB-S systems while leaving the interior beams unprotected. Therefore, this experimental programme applied fire protection strategy recommended in the SCI Publication P288 (Newman *et al.* 2006). The edge beams and the columns were protected to a prescriptive fire-protection rating of 60min using intumescent paint. No fire-proofing material was applied to the interior beams and the slabs.

4.3.2 Design philosophy

4.3.2.1 Series I

The objectives of Series I were to investigate the effects of two parameters, i.e. unprotected interior beams and rotational edge restraint, on the behaviour of composite slab-beam systems under fire conditions. This series included three specimens, denoted as S1, S2-FR-IB, and S3-FR. In this nomenclature FR indicates a rotationally restrained system, while IB indicates the interior beams. Specimens S1 and S3-FR were designed without interior beams and with slightly different test setup to study the effect of rotational edge restraint. S2-FR-IB had two interior beams and the same test setup with S3-FR. Details of the test setup of Series I specimens are presented in **Section 4.4.3**. The dimensions of all specimens were 2.25m long by 2.25m wide, giving an aspect ratio of 1.0. To simulate interior slab panels, all specimens were designed with a 0.45m outstand beyond the edge beams in both directions, as shown in **Fig. 4.4**. In this figure, the notations MB, PSB, and USB denote a protected main beam, a protected secondary beam, and an unprotected secondary (interior) beam, respectively.

Fig. 4.4 Structural layout of the specimens in Series I

The design philosophy is chosen so that the slab is the first member to be failed, subsequently the steel beams and lastly the steel columns if at all. Therefore, all the beams were designed as Class 1 sections according to EN 1993-1-1. The columns were designed as the last component to fail. Thus, they were chosen to be overly strong by using UC 152x152x30.

The use of fabricated sections for all secondary beams was necessary, since there was no Universal Beam section suitable for the required scaling. Once the main and secondary beams had been designed and their sections had been finalized, the connection details were looked into. A common type of steel joints, *viz.* the flexible end plate, was used for both the beam-to-beam and the beam-to-column connections as shown in **Fig. 4.5**. All failure modes specified in EN 1993-1-8 (2005c) were checked in order to avoid any premature failure of the connections due to shear and tension.

Fig. 4.5 Flexible end plate connections

Full-shear composite action between the slab and the downstand beams was achieved by using shear studs of 40mm height and 13mm diameter, with the purpose of avoiding any premature failure due to horizontal shear action, which was not the focus of the test programme. The spacing of shear studs was deliberately designed to be small (80mm centre-to-centre) so that any premature failure at the shear studs did not occur. This was successful, and no fracture of shear studs, or failure caused by incomplete shear interaction, occurred in any specimens.

Due to the 1:4 scaling there was no standard steel decking suitable for the slabs. To protect the heating elements from concrete spalling, the slabs were cast onto a 2mm thick steel sheet with small pre-drilled holes. The contribution of this sheet to the slab load-bearing capacity could be ignored, since the unprotected sheet would debond from the concrete slab as observed in the tests. With regard to heat transfer, the sheet may slow down the heat which transferred from the slab bottom to the top. Therefore, this study cannot give any advice on fire resistance time of the floor systems.

4.3.2.2 Series II

As mentioned before, the properties of the I-section edge beams which may affect the behaviour of the CB-S systems include the grade of steel beams, torsional rigidity GI_t , and bending stiffness in major axis EI_y . The numerical investigations in **Chapter 3** showed that in these three properties, only the bending stiffness in major axis EI_y has significant effect on the membrane behaviour of the systems. Therefore, this parameter was chosen to be studied in Series II which consisted of 5 specimens in total. The design philosophy of Series II was similar to Series I. Thus, it is not repeated here.

Table 4.2 Specimens of Series II

Specimen	Main beam		Protected Secondary Edge Beam		Unprotected secondary beam		Relative stiffness ratio	
		I_y (cm ⁴)		I_y (cm ⁴)		I_y (cm ⁴)		
P215-M1099	W130x130x28	1099	80x80x17.3	215.3	80x80x17.3	215.3	1.00	1.00
P368-M1099	W130x130x28	1099	100x80x19	368.0	80x80x17.3	215.3		1.71

P486-M1099	W130x130x28	1099	102x102x23	486.0	80x80x17.3	215.3		2.26
P215-M1356	UB 178x102x19	1356	80x80x17.3	215.3	80x80x17.3	215.3	1.23	
P215-M2110	UB 203x102x23	2110	80x80x17.3	215.3	80x80x17.3	215.3	1.92	

All the specimens had two interior beams using a fabricated I-section of 80x80x17. The term 80x80x17.3 means the section is 80mm high, 80mm wide and 17.3kg/m mass. P215-M1099 was chosen as the control specimen. Hence, its relative stiffness ratio was considered as unity. The stiffness of PSB of P368-M1099 and P486-M1099 was increased 1.71 and 2.26 times, respectively, compared to that of P215-M1099 to investigate the effect of stiffness of PSB. Similarly, the stiffness of the main beam of P215-M1356 and P215-M2110 was increased 1.23 and 1.92 times, respectively, compared to that of P215-M1099, to study the effect of stiffness of MB. The section of USB was kept constant in all specimens.

Fig. 4.6 shows the structural layout of the specimens in Series II. The slabs were 2.25m long and 2.25m wide with 0.45m overhang around the four edges. Along each edge, five M24 bolts were used. Half of the bolt length was cast with the slab, while the other half was outside the edges. The locations of these bolts were fixed by using 8mm thick steel plate along the slab edges. A rectangular hollow section beam system was attached around the slab edge to simulate the inplane restraint. Rotational restraint was applied by a beam system placed on top of the overhang. The purpose of these bolts was to simulate accurately the boundary conditions of interior slab panels. The interior slab panels should be rotationally restrained around the four edges and could only have horizontal straight movement as explained below. Details of test setup of Series II are given in **Section 4.4.4**.

Fig. 4.6 Structural layout of the specimens in Series II

It is questionable whether the slab edges would translate straight if the slab is considered as an interior slab. Considering a composite steel-framed building, there would be two fire scenarios. *The common scenario* is the situation where the fire heats up the whole soffit of the floor (**Fig. 4.7(a)**). In this situation, displacement along the slab edges can be outward caused by thermal expansion or inward resulted from tensile membrane action mobilised at a later stage. However, the common edges between two interior slab panels must translate straight to ensure displacement compatibility. Therefore, in this case the slab edges can only move outwards and inwards straight. *In the second fire scenario*, if the fire is localised within one apartment as shown in **Fig. 4.7(b)**, the ‘cool’ zones around the heated zone provide inplane restraint to the outward movement of interior slab panels due to thermal expansion, causing restrained forces. The stiffness of inplane restraint of the ‘cool’ zone depends on many factors, such as number and the dimensions of unheated spans, the locations of shear walls, etc. These factors are impossible to be investigated experimentally.

Fig. 4.7 Two fire scenarios

In this research, the author aimed to model accurately interior slab panels in the first fire scenario as shown in **Fig. 4.7(a)**. Due to the two restraint beam systems, all specimens were considered as the interior slab panels which were rotationally

restrained and could move outwards and inwards in straight lines at the four edges. Test results shown in **Chapter 6** indicate that this research purpose had been achieved.

4.3.3 Test load

In this experiment, the floor assemblies were loaded uniformly based on a predetermined load level and subsequently heated from the bottom. The load level was defined as the test load divided by the conventional yield-line load at ambient temperature. For normal design at ambient temperature, composite slabs are assumed to act as one-way spanning slabs. Therefore, the yield load of the specimens with interior beams (S2-FR-IB and Series II) is calculated based on a slab panel of 0.75m x 2.25m. For the specimens without interior beams (S1 and S3-FR), the yield load is determined based on a slab panel of 2.25m x 2.25m. Considering combined failure modes for a beam-slab system, there are three possible failure modes as shown in **Fig. 4.8**.

Fig. 4.8 Combined failure modes of a slab-beam system

Based on the assumed yield line patterns, the yield-line load can be obtained using yield line theory as shown in **Table 4.3**. It can be seen that the third failure mode gives the smallest ultimate strength. Therefore, this collapse mechanism is considered to be the correct failure mode and the yield line load at ambient temperature is 8.3kN/m^2 for S1 and S3-FR, and 36.6kN/m^2 for S2-FR-IB and Series II. However, in order to compare the test results, all specimens were loaded with the same value, of 15.8kN/m^2 , which corresponded to a load ratio of 1.9 for S1 and S3-FR, and 0.43 for S2-FR-IB and Series II.

Table 4.3 Yield-line failure load of the specimens

<i>Specimen</i>	<i>Mode</i>	l_x <i>mm</i>	l_y <i>mm</i>	h <i>mm</i>	ρ_x <i>%</i>	ρ'_x <i>%</i>	l_1 <i>mm</i>	l_2 <i>mm</i>	w_u <i>kN/m}^2</i>
without interior beams	1	2250	2250	55	0.19	0.0	1125	0	57.4
	2	2250	2250	55	0.21	0.0	0	1125	23.5
	3	2250	2250	55	0.21	0.0			8.3
with interior beams	1	2250	750	55	0.19	0.0	375	0	517.0
	2	2250	750	55	0.21	0.0		1125	59.8
	3	2250	750	55	0.21	0.0		375	36.6

4.3.4 Construction process

a) Construct steel frame

b) Formwork and reinforcement

c) Casting

d) Curing and painting

Fig. 4.9 Construction sequences

Fig. 4.9 describes the construction sequence. The whole process commenced with the build-up of steel frame and reinforcement for the concrete slab. This was followed by placement of formwork. The installation of thermocouples for temperature measurements was done after the entire steel cage had been formed. After casting, thermocouples measuring the steel beam temperatures were installed onto the steel section, i.e. the top and bottom flanges and the beam web. The specimens were cured for 28 days. The edge beams and columns were then protected to a prescriptive fire-protection rating of 60min using intumescent paint.

4.4 Test Setup and Test Procedure

The test setup was designed in such a way to achieve the objectives as well as to overcome many constraints due to equipment, laboratory space and most importantly, safety issues. The feasibility and ease during the setting up also require careful consideration.

4.4.1 Loading system

Composite slab-beam systems normally support uniformly distributed load which consists of permanent and variables actions. Therefore, uniformly distributed load (UDL) should be simulated accurately in any experimental programme. In some full scale fire tests (Zhao *et al.* 2008; Stadler *et al.* 2011), sand bags were used to simulate external loads. However, sand bags could not be used in the author's experiment due to two reasons. Firstly, because of scaling, there was not enough area

to dispose the sand bags. Secondly, when the slab underwent large deformation, the sand bags might move, causing damage to the electric furnace.

Therefore, a loading scheme was specially designed based on existing laboratory constraints to simulate UDL. A 50-ton hydraulic jack held by a reaction steel frame was used to load the specimens. The load from the hydraulic jack was distributed equally to twelve points by means of a set of loading trees, which consisted of three RHS beams and four triangular steel plates (**Fig. 4.10**). The loading system has already been used at Nanyang Technological University (Pham 2012) and was capable of simulating UDL. Twelve loading points were disposed in such a way that each loading point loads an equal area of $1/12$ times of the interested zone, $2.25\text{m} \times 2.25\text{m}$ (**Fig. 4.11**).

Fig. 4.10 Loading system

Fig. 4.11 Positions of 12 loading point

The loading system was painted by intumescent paint to avoid any strength degradation during testing in fire. Between steel plates and steel rods, ball and socket joints were used to maintain verticality of the loading system when the specimens deformed excessively. Steel plates were used under all the loading points to prevent unexpected punching failure.

In order to verify the accuracy of the loading system, finite element analysis using SAP2000 (2010) were conducted. The models are shown in **Fig. 4.12**, in which the loaded area of $2.25\text{m} \times 2.25\text{m}$ is subjected to a UDL of 15.8kN/m^2 or 12 point loads of 6.67kN each.

Fig. 4.12 Verification of accuracy of the loading system

The results summarised in **Table 4.4** indicate that there are only small discrepancies between these two methods of loading. Therefore, the 12-point loading system can represent the uniformly distributed load well.

Table 4.4 Comparison of two methods of loading

	<i>Uniform load</i>	<i>12-loading point</i>	<i>Error</i>
Central deflection of slab	5.4mm	5.6mm	3.7%
Midspan deflection of main edge beam	1.8mm	1.8mm	0.0%
Midspan deflection of secondary edge beam	2.7mm	2.8mm	3.7%
Axial force in columns	20.00kN	20.01kN	0.05%
Bending moment at the top of columns	3.10kNm	3.28kNm	5.8%

4.4.2 Test rig

a) Design of test setup

Pin connections

b) Final test setup

Fig. 4.13 Typical test setup

The test setup is shown in **Fig. 4.13** to which the following description makes references. The system consisted of an electrical heating furnace (1), a hydraulic jack (2) and an external reaction steel frame (3). The vertical hydraulic jack was used as a loading device attached to the steel frame. The load from the jack was distributed equally to twelve point loads by means of the loading system (4). The heating furnace was located beneath the slab and supported by two steel beams (5) resting on the laboratory floor. A specimen (6) which consisted of a composite beam-slab system and four 1.15m long I-section columns was connected to four circular hollow section (CHS) columns (7) through extended end plates and then seated onto four column bases (8). Each circular column was fixed onto the laboratory floor as a pinned connection. Vertical load was measured by an annular load cell with 300kN capacity (9) placed between the jack and the loading system. A rocker bearing (10) was inserted between the load cell and the hydraulic jack to ensure verticality of the applied force. To simulate rotational restraint along the four edges, a system consisted of four rectangular hollow section beams (11) was placed on top of the slab outstand and fixed to the columns via stiffeners (12).

The use of circular pin-ended columns allowed the specimen to move horizontally without any degree of restraint from the columns. More importantly, due to the use of the circular sections, vertical reaction forces in four perimeter columns can be determined and checked against the applied load, which is an important objective of Series I.

4.4.3 Test setup of Series I

Fig. 4.14 shows the test setup of Series I. All gaps between the furnace and the specimen were filled with thermal superwool blanket to reduce heat loss. The test setup was changed for S2-FR-IB and S3-FR to study the effect of peripheral rotational restraint on the slab-beam behaviour. A restraint system consisting of four 160x100x6 rectangular hollow section (RHS) beams was placed on top of the slab outstand and fixed to the reaction frame via two triangular stiffeners (**Fig. 4.14(b)**). It is assumed that, for all specimens, reinforcement continuity over the supporting protected edge beams and the 0.45m wide outstand provided little rotational restraint, since there was only one layer of shrinkage reinforcement. Therefore, S1 can be considered as a rotationally unrestrained slab since its outstand could curl upwards freely when subjected to loading. Slabs S2-FR-IB and S3-FR are considered as rotationally restrained by the additional RHS beam system. In Series I, there was no horizontal restraint along the slab edges.

a) S1 (with no rotational restraint)

b) S2-FR-IB & S3-FR (with rotational restraint)

Fig. 4.14 Difference in boundary conditions of Series I

All the specimens were loaded to the same value of 15.8kN/m^2 in order that comparisons of the test results could be conducted. This value of loading corresponded to a load ratio of 1.97 for S1 and S3-FR, and 0.43 for S2-FR-IB based on the yield-line load at ambient temperature. The yield-line loads were calculated as 8.02kN/m^2 for S1 and S3-FR, and 36.7kN/m^2 for S2-FR-IB, which were greatly different. This is because the yield-line load at ambient temperature of S2-FR-IB, which had two interior beams, is significantly higher than those of S1 and S3-FR, which had no interior beams.

Using a load ratio of 1.97 for S1 and S3-FR is very high and unrealistic. This may cause cracks at the early stage of the tests – bending stage. However, this did not affect significantly the results at tensile membrane stage, at which the concrete experienced severe cracks and the load-bearing capacity of the slabs was mainly contributed by reinforcing mesh. On the other hand, this load ratio used for S1 and S3-FR aimed to enable to compare the results with specimen S2-FR-IB.

4.4.4 Test setup of Series II

The overall test setup of Series II is shown in **Fig. 4.15**. The specimens were setup together with two restraint beam systems. *The first system* was the rotational restraint system which consisted of four $160\times 100\times 6$ rectangular hollow section (RHS) beams placed on top of these specimens and fixed to the reaction frame via two triangular stiffeners. Similarly with Series I, it is assumed that reinforcement continuity over the supporting protected edge beams and the 0.45m slab outstand provided very little rotational restraint, since there was only one layer of shrinkage reinforcement placed inside the slabs.

The second system was the ‘so-called’ in-plane restraint system, which also consisted of four $160\times 100\times 6$ RHS beams. This system was fixed to four slab edges via five M24 bolts along each edge at a spacing of 750mm (**Fig. 4.6**). The in-plane restraint system was also connected to the rotational restraint system by a different line of bolts (**Fig. 4.16**). These two systems aimed to simulate accurately the continuity of interior slab panels. The in-plane restraint system allowed the slab edges to translate

inwards or outwards in straight lines, while the rotational restraint system applied flexural restraint on the slab edges. Test results presented in **Section 6.3** indicate that this research purpose has been achieved. It is worth noting that there was a 20mm gap between the in-plane restraint system and the furnace walls to avoid any load taken up by the furnace walls. The gap was filled by insulation material to avoid heat loss.

Fig. 4.15 Test setup of Series II

Fig. 4.16 Connection between the in-plane and rotational restraint beam systems

4.4.5 Sequence of test setup

For ease of setting-up and safety requirements, the sequence of test set-up was chosen as described in **Fig. 4.17**. Firstly, a specimen was lifted up and put on top of the furnace. After fixing the bolts, thermocouples were attached to the slab top surface. The loading system was placed at the correct position, and then LVDTs were assembled. All wires were connected to a computer and a trial test was conducted.

4.4.6 Test procedure

Before conducting a test, the top surface of the slab was painted with a thin coat of white-wash to provide better crack observation. The instrumentation was calibrated and initialised. A small load was first applied to check the test setup and if the instrumentation had functioned as planned.

Transient-state heating was used since the research objective was to obtain load-deflection-temperature characteristics of the CB-S systems as temperature increased. Therefore, the specimens were first loaded to the predetermined value as mentioned in **Section 4.3.3**, and then heating was applied. The furnace temperature was increased while the load was manually maintained constant. When the failure had been identified, the test was terminated. The cooling phase was allowed to happen naturally.

4.5 Summary

This chapter introduced the details of the preparatory works including the design and setup of a new electric furnace, and the design philosophy of the specimens. The test setup of two series and the test procedure were presented.

Experimental results, observations, numerical validation, and discussions of two test series are presented in **Chapters 5 and 6**.

Step 1: Lift specimen on top of furnace

Step 2: Attach thermocouples on top surface

Step 3: Place the loading system

Step 4: Connect the steel beam of frame

Step 5: Place instrumentations

Step 6: Connect to computer and test

Fig. 4.17 Sequence of test setup

CHAPTER 5 EXPERIMENTAL AND NUMERICAL STUDIES OF SERIES I

5.1 Introduction

There has been a recent interest in membrane behaviour of integrated floor assemblies, such as FRACOF fire test (Zhao *et al.* 2008) in 2008, Munich fire test (Stadler *et al.* 2011), Zhang *et al.* (2009), and Wellman *et al.* (2011). Extensive review in **Chapter 2** shows that although these previous studies provide valuable insight of the behaviour of composite floor assemblies in fire, the effects of unprotected interior secondary beams and of rotational restraint on the development of TMA are still not clearly identified. On the other hand, it has been numerically shown in **Chapter 3** that although suffering a great reduction of strength and stiffness under severe fire, unprotected interior beams have beneficial effect on the system behaviour because with the presence of the interior beams helps to stabilise the floor system. Therefore, Series I was designed to study these effects experimentally.

This chapter is organized into six sections. **Section 5.2** describes material properties of test specimens and instrumentation. Performance and observations of the specimens are summarised in **Section 5.3**. In **Section 5.4**, numerical simulations are performed to model the test results. Once the model has been validated, numerical assessments and discussions are conducted and presented in **Section 5.5**. **Section 5.6** provides conclusions of this test series.

The *objectives* of Series I are to:

- (1) study the effect of unprotected interior beams on TMA;
- (2) investigate the effect of rotational edge-restraint on TMA;
- (3) capture failure modes and deflection shapes of the CB-S systems;
- (4) investigate load distribution mechanism from the slab to the columns;
- (5) provide experimental results to validate numerical models.

5.2 Test Specimens

Series I consisted of three specimens, namely, S1, S2-FR-IB and S3-FR. In this nomenclature FR indicates a rotationally restrained system, while IB indicates interior beams. Specimens S1 and S3-FR were designed without interior beams, while S2-FR-IB had two interior beams. Specimens S1 and S3-FR were designed with similar parameters, but their test setup were changed to study the effect of rotational edge-restraint (**Section 4.4.3**). Comparing the test results of S2-FR-IB and S3-FR could highlight the effect of unprotected interior beams.

Fig. 5.1 Structural layout of S1 and S3-FR

Fig. 5.2 Structural layout of S2-FR-IB

The dimensions of all specimens were 2.25m long by 2.25m wide, giving an aspect ratio of 1.0. To simulate interior slab panels, all specimens were designed with a 0.45m outstand beyond the edge beams in both directions, as shown in **Figs. 5.1** and **5.2**. In these figures, the notations MB, PSB, and USB denote a protected main beam, a protected secondary beam, and an unprotected secondary (interior) beam, respectively.

All the beams were Class 1 sections according to EN 1993-1-1 (2005a). The use of fabricated sections for all secondary beams was necessary, since there was no Universal Beam section suitable for small scale tests. Full-shear composite action between the slab and the downstand beams was achieved by using shear studs of 40mm height and 13mm diameter. Spacing of shear studs was deliberately designed to be small (80mm centre-to-centre), in order to avoid premature failure at the shear studs. This was successful, and no fracture of shear studs, or failure caused by incomplete shear interaction, occurred in any of the three tests. A common type of steel joint, *viz.* the flexible end plate, was used for both beam-to-beam and beam-to-column connections as shown in **Fig. 5.3**.

Fig. 5.3 Beam-to-column and beam-to-beam connections

5.2.1 Material properties

Material properties were tested at ambient temperature, which included cylinder compressive tests for concrete, tensile tests for steel beams and reinforcement.

Material properties at elevated temperatures were then deducted from the results at ambient temperature by using the widely-accepted Eurocode models (**Section 2.2**).

Concrete slabs

The target thickness of the slabs was 55mm. Shrinkage reinforcement mesh with a grid size of 80mm x 80mm and a diameter of 3mm (giving a reinforcement ratio of 0.16%) was placed about 38mm below the slab top surface, which was ensured by the 40mm x 40mm concrete supporting blocks. These concrete blocks were placed at a spacing of 320mm x 320mm. The 0.16% reinforcement ratio was well within the allowable range specified in EN 1994-1-1 (0.2% for un-propped construction and 0.4% for propped construction). The mesh was continuous across the whole slab, with no lapping of mesh. The specimens were cast using ready-mixed concrete, with the aggregate size ranging from 5 to 10mm, to enable adequate compaction during placement. Specimens S1 and S2-FR-IB were cast from a single concrete batch, whereas S3-FR was from a separate batch. Thus, the values of mean compressive strength f_{cm} slightly differ.

Six concrete cylinders of 150mm in diameter and 300mm in length were tested 28 days after casting, to determine the mean compressive strengths f_{cm} , and then the corresponding elastic modulus E_{cm} and the mean tensile strength f_{ctm} were calculated in accordance with EN 1992-1-1 (2004b), as shown in Eqs. (5.1) and (5.2). **Table 5.1** summarises the properties of the concrete slabs.

$$E_{cm} = 22 \left[(f_{ck} + 8) / 10 \right]^{0.3} \quad (f_{ck} \text{ in MPa}) \quad (5.1)$$

$$f_{ctm} = 0.3 \times f_{ck}^{(2/3)} \leq C50 / 60 \quad (5.2)$$

Table 5.1 Properties of concrete slabs – Series I

	h_s mm	d_x mm	d_y mm	f_{ck} MPa	f_{ctm} MPa	E_{cm} GPa	f_y / f_u MPa	Ultimate strain %	E_s GPa
S1	55	38	35	36.3	3.3	34.4	543/771	22.2	180
S2-FR-IB	55	38	35	36.3	3.3	34.4	543/771	22.2	180
S3-FR	58	40	37	31.3	3.0	33.2	689/806	14.8	203.4

Reinforcement

The use of 3mm diameter shrinkage reinforcement mesh was necessary due to small scale tests. Tensile tests were conducted on five reinforcement samples. For S1 and S2-FR-IB, the average yield strength and ultimate strength were 543MPa and 771MPa, respectively; while the corresponding values for S3-FR were 689MPa and 806MPa. The average elastic modulus was determined as 180GPa for S1 and S2-FR-IB, and 203.4GPa for S3-FR. These values are summarised in **Table 5.1**, and a typical stress-strain relationship is shown in **Fig. 5.4**. Since a strain gauge was unable to be mounted on 3mm diameter rebars, the elastic modulus was determined based on the data obtained from an extensometer. Also, due to cold-worked steel, the stress-strain curve did not show a clearly defined yield point. Therefore, the 0.2% proof stress was taken as the yield strength.

Fig. 5.4 Typical stress-strain relationship of reinforcement – S1 & S2-FR-IB

Structural steels

Tensile coupon tests were conducted at ambient temperature to obtain the elastic modulus and yield strength of the beams. For each types of the I-section used, four coupons were tested, two from the flanges and two from the web. For each coupon, two sources of readings were obtained, *viz.* two strain gauges and one extensometer. **Table 5.2** summarises both the average results from the tensile tests and the measured geometrical properties of the beams, while **Fig. 5.5** shows typical stress-strain curves for the coupons from the main beams and protected secondary beams. It should be noted that the columns were taken from the same batch for all of the tests.

Fig. 5.5 Typical stress-strain relationship of structural steel beams

Table 5.2 Properties of I-section beams – Series I

Specimen		Depth h_{sb} (mm)	Width b_f (mm)	Thickness		Yield strength f_y (MPa)	Ultimate stress f_u (MPa)	Elastic modulus E_s (GPa)
				Web (mm)	Flange (mm)			
S1 & S2-FR-IB	MB	131	128	6.96	10.77	302	437	197.5
	PSB & USB	80	80	9.01	9.14	435	533	206.9
	UC	157.6	153	6.90	9.70	321	489	196.7
S3-FR	MB	131	128	6.97	11.03	307	462	211.4
	PSB & USB	80	80	10.26	10.02	467	588	210.6
	UC	157.6	153	6.90	9.70	321	489	196.7

5.2.2 Instrumentation

5.2.2.1 Acquisition of deformation and temperature measurements

A free-standing outer frame was fabricated and placed around the furnace as a reference support to measure necessary displacements. Twelve linear variable differential transducers (LVDT) were used to measure deflections of the slab and the beams, as shown in **Fig. 5.6**. S1 (which did not have the rotational restraint system) had one load cell (LC1) at its centre to measure forces from the hydraulic jack. For S2-FR-IB and S3-FR, three load cells were applied with one at the slab centre to

record forces from the jack, and two below the stiffeners to measure reaction forces from the restraint beam system (**Fig. 4.14(b)**).

Fig. 5.6 Arrangement of LVDTs and load cells

Fig. 5.7 Arrangement of thermocouples

Each specimen was instrumented with 21 K-type thermocouples to capture temperatures at various locations of the slab and beams, as shown in **Fig. 5.7**. Each member's temperature profile was monitored at two or more locations to check temperature uniformity. The temperature was measured at the slab bottom and top surfaces and at the reinforcing mesh level, denoted as SB, ST and S respectively. The beam temperature was recorded at the bottom and top flanges of I-section, and in the web. The thermocouples for beams were denoted as MB, MT and MW for main beams. Similarly, the notations PB, PT and PW were used for secondary edge

beams, and UB, UT and UW for unprotected secondary beams. The furnace air temperature was also monitored by four thermocouples which were positioned at two side heating panels and two interior positions within the furnace.

5.2.2.2 Measurement of forces

To verify credibility of the test setup, the specimens of Series I were mounted with measuring devices both internally and externally. External load from the hydraulic jack was measured by an annular compression load cell with 300kN capacity placed between the jack and the loading system. Vertical reaction forces in four circular supporting columns could be determined through readings from four strain gauges mounted on opposite external surfaces of the columns (**Fig. 5.8**). This method was used by Tan et al. (2010) and it has shown the accuracy in determining the applied forces.

Fig. 5.8 Calculation of reaction forces by strain gauges

The internal forces at the section where the strain gauges were mounted could be evaluated by steel strains and cross-sectional properties by Eq. (5.3).

$$N_1 = E_s \times A_s \sum_{i=1}^4 \varepsilon_i / 4 \quad (5.3)$$

where A_s is area of the section; ε_i are the readings from the strain gauges SG-1,2,3,4.

5.3 Test Results and Observations

At the end of the loading phase prior to heating, the slab's central initial deflections (measured by L1 and L2 in **Fig. 5.6**) were very small, at 2.5mm, 2.7mm and 3.7mm for S1, S2-FR-IB and S3-FR, respectively. Therefore, the graphical presentation of results only shows the deflections during the heating phase. The deflection of S1 seemed to be small with such a high load ratio (1.97 for S1). This could be explained by the contribution of the 2mm steel decking at room temperature when the decking still maintained its full strength and stiffness. On the other hand, it is somehow hard to explain why the deflection of S2-FR-IB was greater than that of S1. It may be due to inaccuracy of measurement since the absolute values of deflection was quite small, i.e. only 2.5mm for S1 and 2.7mm for S2-FR-IB. In **Section 5.3.1**, all the results in both loading and heating stages are shown, because it is necessary to show load-time relationships during the tests.

The tests were terminated when “failure” occurred. This was defined as the time when either:

- (1) Full-depth cracks with the crack width of about 10mm in the vicinity of the edge beams or failure of compression ring can be observed clearly; or
- (2) There was a significant drop in the mechanical resistance, and the hydraulic jack could no longer maintain the load level (violation of criterion “R”).

These failure criteria are different from those used in the numerical studies (**Chapter 3**) which are defined based on EN 1363-1 (2012). The failure criteria in the numerical studies are used for analysis purpose. To understand the structural behaviour, in experiment, the specimens should be tested as far as possible.

5.3.1 Total reaction forces against the applied load

At the loading phase, force from the hydraulic jack was increased slowly up to a predetermined value of 80kN, which was equal to 15.8kN/m^2 applied on the interested slab zone, 2.25m x 2.25m. When the force reached 80kN, the furnace was turned on and heating started.

Fig. 5.9 shows a comparison between the force measured by the load cell at slab centre (LC1) and the total reaction forces measured by the strain gauges mounted on the circular columns for S1. It was expected that the load was kept constant during heating. Unfortunately, in S1 test it was not possible to maintain the load constant during the heating phase due to oil leaks. Although the oil leaks occurred, discrepancy between two results was very small, only 5%, showing the credibility of the test setup.

Fig. 5.9 Comparison of total reaction forces and load cell – S1

Figs. 5.10 – 5.11 show the comparisons between the forces measured by the load cells and the total reaction forces calculated from the data of strain gauges mounted on the circular columns for S2-FR-IB and S3-FR, respectively. LC1 denoted the load cell at the slab centre; LC2 denoted the two load cells placed below the stiffeners to measure reaction forces from the restraint beam system.

At the beginning of S2-FR-IB and S3-FR tests, the slab outstand curled upwards resulting in the restraint forces measured by the two load cells LC2. As the slab deflection increased, the restraint forces increased with the maximum value of 5.5kN for S2-FR-IB and 8.4kN for S3-FR. These vertical forces were resulted from the rotational restraint beam system placed at the overhang and on top of the specimen. However, after 38.9min (27.1min of heating) for S2-FR-IB and after 19min (5min of heating) for S3-FR, cracks started to appear in the vicinity of the edge beams. This led to a separation of the slab outstand and the rotational restraint beams. Therefore, the value measured from LC2 reduced gradually to zero.

Fig. 5.10 Comparison of total reaction forces and load cells – S2-FR-IB

Fig. 5.11 Comparison of total reaction forces and load cells – S3-FR

In principle, the sum of the forces from the hydraulic jack and the restraint forces (measured from two load cells LC2) should be equal to the total reaction forces from the four steel columns. For S2-FR-IB, from the beginning of the test to 86min, the two results are quite consistent. However, after 86min, cracks over the protected main beams developed significantly, leading to a sudden drop of the load. Since there was no clear failure observed, the author tried to maintain the load to see if the slab deflection could increase. After 95.8min, failure had occurred, and therefore the load was reduced. Generally, the two results agreed very well. It can be concluded that the test setup was very reliable because the internal forces calculated from the

strain gauges and the external load measured from the load cells were very consistent. The discrepancy is only 3.8%, 2.3% and 6% for S1, S2-FR-IB and S3-FR, respectively.

Table 5.3 Discrepancy in comparison of total reaction forces and load cell at the end of the loading phase

<i>Specimen</i>	<i>Sum of total reaction forces (kN)</i>	<i>Sum of load cells (kN)</i>	<i>Discrepancy (%)</i>
S1	76.75	79.8	3.8%
S2-FR-IB	83.02	85	2.3%
S3-FR	86.69	92.2	6.0%

5.3.2 Load distribution to columns

Figs. 5.12 – 5.14 show load distribution to four supporting steel columns for S1, S2-FR-IB and S3-FR. The columns were shown to support equal part of load during both loading and heating stages with only a small variation. After 90min in S1 test and 40min of S2-FR-IB test, when cracks developed significantly, the load was redistributed to the columns. The load redistribution depended on the dispersion of concrete cracks. Therefore, it can be concluded that load is distributed equally to the columns during both loading and heating stages, but there is a redistribution of the load when concrete cracks appear severely in the vicinity of the edge beams.

Fig. 5.12 Load distribution to the columns – S1

Fig. 5.13 Load distribution to the columns – S2-FR-IB

Fig. 5.14 Load distribution to the columns – S3-FR

5.3.3 Temperature distributions in the slabs

5.3.3.1 Temperature distributions in horizontal directions

Temperatures measured by the thermocouples at three different locations of the slabs but at the same cross-sectional height, i.e. at reinforcing mesh (denoted as S1, S2, and S3), at bottom surfaces (denoted as SB1, SB2, and SB3) and at top surfaces (denoted as ST1, ST2, and ST3), were compared to check whether the temperature was distributed uniformly. Positions of thermocouples are indicated in **Fig. 5.7**. All thermocouples functioned well during the tests, except SB1 and S1 in S3-FR test.

Fig. 5.15 indicates that for all three tests, the temperatures measured by the thermocouples at the same cross-sectional height were very consistent. Therefore,

the assumption that temperature only varies across the slab thickness and does not vary in the horizontal directions is validated. This also gives credibility and confidence to the fire test set-up.

(a) S1

(b) S2-FR-IB

(c) S3-FR

Fig. 5.15 Temperature distributions in horizontal directions of the slabs – Series I

5.3.3.2 Temperature distributions across the slab thickness

Fig. 5.16 shows temperature of the furnace air and those measured across the slab thickness. These are the averages of values recorded by three thermocouples at different locations. For example, the temperature at the reinforcing mesh is the average of the values measured by thermocouples S1 to S3 (**Fig. 5.7**).

Development of air temperature was shown to be similar up to 45min of heating. Beyond this time, there was a small but acceptable deviation of 74°C between the maximum temperature of S1 (909°C) compared to that of S2-FR-IB (983°C). This was because as the temperature increased, S1 which allowed the slab to curl upwards experienced greater heat loss than S2-FR-IB. Test S3-FR was terminated after 59min of heating when failure had been identified due to failure of compression ring at the slab corners.

Fig. 5.16 Temperature development through the slab depth

Temperature at the slab bottom surface was consistent as expected. However, temperatures at the reinforcing mesh and at the slab top surface diverged towards the end of the tests, since the temperatures at these locations depended not only on the air temperature, but also on development of cracks in the concrete slab. As the slab deflected, the crack pattern was different.

On the other hand, the temperatures at the mesh level and at the slab's unexposed surface increased slowly, progressing from 23°C to only 120°C within the first 40min. This was due to moisture release from inside the concrete slab, which slowed down the temperature increase rate during the initial stage. When the free water in the concrete had almost evaporated, the temperature increased at a greater rate.

5.3.4 Slab deflection

Fig. 5.17 shows the applied load-time relationships. The designated constant value of the load for all specimens was 15.8kN/m². Unfortunately, in S1 it was not possible to maintain a constant load level during the heating phase, caused by oil leaks during the test. Load had to be reapplied manually to keep it constant. In S2-FR-IB and S3-FR, the load was maintained as expected. At failure, the load decreased very fast, and the hydraulic jack could no longer maintain the load level.

Fig. 5.17 Load – time relationship

The mid-span deflection against time of the slabs, together with the corresponding failure point was plotted in **Fig. 5.18**. The maximum slab deflections were 131mm, 177mm and 115mm for S1, S2-FR-IB and S3-FR respectively. However, the failure times were very similar, at 85.8min for S1 and 84min for S2-FR-IB. S3-FR failed very early, at only 45min of heating due to 'brittle' failure as explained in **Section 5.3.7**.

Due to oil leaks, S1 experienced a deflection rebound between 30min and 45min of heating. Although the mechanical load reduced significantly during this period, from 15.8kN/m^2 to 8.16kN/m^2 (**Fig. 5.17**), the deflection stayed constant as shown in **Fig. 5.18**. This indicated that thermal effects (thermal bowing and thermal expansion) are the main factors in controlling the structural response under fire conditions, rather than the mechanical load.

Fig. 5.18 Mid-span slab deflection vs. time

5.3.5 Behaviour of steel frames

5.3.5.1 Protected main and secondary edge beams

Mid-span deflection and temperature against time

Fig. 5.19 shows mid-span deflection and temperature development of the protected main and secondary edge beams against time. Temperatures at the beam bottom flanges and at the beam web were very close, while temperature at the beam top flange was slightly lower. This was because the beam depth which affects greatly the beam temperature distribution was small, only 131mm for MB and 80mm for PSB.

The mid-span deflection of the beams was measured by L8 and L11 for the main beam (MB) and L5 and L10 for the protected secondary beam (PSB). Both results for each beam were plotted to verify the beam behaviour. L8 and L11 gave similar deflections with only a slight discrepancy. The same trend was observed for L5 and L10. This indicated that in all the tests, the beams did behave symmetrically. In S2-

FR-IB test, beyond 62min of heating, L5 was inclined, and thus its data could not be used anymore.

(a) S1

(b) S2-FR-IB

(c) S3-FR

Fig. 5.19 Deflections and temperatures vs. time of protected edge beams – Series I

However, in S3-FR, the deflection results from L8 and L11 were different, since in S3-FR severe cracks appeared at a very early stage (just 20min after heating) directly above the main beams. These two cracks above both main beams developed severely and differently. Therefore, the two sets of the results were different. The reasons for appearance of these early cracks are explained in **Section 5.3.5**. It should be noted that the beam deflections were measured by the LVDTs from the top surface of the concrete slab.

Comparison of mid-span deflections

Fig. 5.20 shows the average mid-span deflection of the main beams (MB) and the protected secondary beams (PSB), respectively. For S1 and S2-FR-IB, the deflection-temperature curve can be divided into three phases. In Phase 1, up to 25min of heating (temperature of about 300°C for MB and 250°C for PSB) the beams deflected downwards linearly caused by thermal bowing. Phase 2 was marked with a constant deflection rate up to about 420°C for both cases, followed by an increased rate up to 700°C for MB and 650°C for PSB. The constant deflection rate can be explained by the decrease of thermal bowing as the temperature propagated through the slab cross-section. Beyond 450°C the beams gradually lost protection, causing a greater deflection rate. In Phase 3, when the bottom flange temperature was about 700°C for MB and 650°C for PSB, the beams showed ‘run away’ behaviour since at this temperature both the strength and stiffness of the beams had reduced significantly.

Fig. 5.20 Comparison of mid-span deflections of the edge beams

However, in S3-FR the deflection-temperature curve for the main beam did not follow this trend. After Phase 1, the main beam of S3-FR already showed significant deflection. This was because in S3-FR severe cracks appeared at a very early stage (just 20min after heating) directly above the main beam. Therefore, composite action between the main beam and the slab could not be maintained, leading to inaccurate measurements of deflections, which were obtained from the top of the concrete slab placed directly above the main beam. In contrast, part of the concrete slab directly above the beams in S1 and S2-FR-IB, as well as that above the protected secondary beams in S3-FR, was fully composite with the beams. Therefore these recorded results were reliable.

After cooling, in all the tests it was observed that local buckling of the beam bottom flanges had not occurred. This could be due to partial extension of the beams through the flexible end plate connections, in addition to the overall expansion of the slab system.

5.3.5.2 Steel columns and connections

Fig. 5.21 shows the deformed shapes of the protected steel columns after cooling. They indicate that the columns were subjected to biaxial bending due to pulling-in of the edge beam ends at large deflections. Buckling of the column flanges was observed in S2-FR-IB, but was not detected in S1 and S3-FR. This is possibly because in S2-FR-IB, there was greater mobilization of TMA due to the presence of two unprotected interior beams. This caused greater biaxial bending in the main beams. In turn, these bending moments induced greater compression forces for parts of the columns.

None of the connections failed or fractured in any of the three tests, during either the heating or cooling phases. This seems to indicate that, at least where there is limited axial restraint, if the connections are designed accurately in accordance with EN 1993-1-8 (2005c), and are protected in fire to the same rating as the connected members, the composite slabs can mobilise TMA without any connection failure during the period.

Fig. 5.21 Deformed shapes of columns after testing

5.3.6 Development of crack patterns

The crack pattern developments observed during the tests are redrawn from **Fig. 5.22** to **Fig. 5.24**. In these figures, the heating times at which the cracks occurred are indicated, together with the corresponding temperature in the reinforcing mesh and the corresponding mid-span slab deflection. The symbol (*) denoted the time when compression ring formed.

Cracking noises were heard approximately from 10 to 12min after heating. This was because at this time, diagonal cracks through the slab near the beam-to-column joints started to appear consecutively at the four corners. These cracks developed gradually and opened up through the slab thickness. They occurred in all the tests, and were due to biaxial bending of the slab outstand. At the slab corners, part of the outstand was in biaxial bending, but was restrained by the columns. Therefore, these cracks consistently formed at an angle ranging from 30° to 45° to the slab edges. Water vapour started to emit from these cracks at this time.

After the appearance of the corner cracks, additional cracks formed along the protected edge beams of the slabs. However, the sequence differed between the tests. In S1 and S3-FR, cracks appeared simultaneously along the edge-beam lengths, followed by cracks at the slab centre. In S2-FR-IB, more severe cracks were

observed, first above the main beams and then in the vicinity of the protected secondary beams. Cracks along the unprotected secondary beams appeared near to the end of the test. During S2-FR-IB test, minor cracks appeared at its slab mid-span. These cracks had closed up after the test, possibly due to the rebound of deflection during cooling. These different sequences of crack development can be attributed to the different load paths from the slabs to the protected edge beams, as explained in **Section 5.3.7**.

Fig. 5.22 Development of crack patterns – S1

Fig. 5.23 Development of crack patterns – S2-FR-IB

Fig. 5.24 Development of crack patterns – S3-FR

(*) *Compression ring formed*

For S1 and S3-FR, after 30min of heating, the compression ring began to form when the mesh temperature had reached about 100°C. The corresponding deflections were 42mm and 52mm, equal to 0.8 and 0.95 of the slab thickness respectively. For S2-FR-IB, after 50 min of heating the compression ring began to form at a mesh temperature of 220°C. The corresponding deflection was 52mm, or 0.95 of the slab thickness. It is obvious that S2-FR- IB, which had interior beams, entered the tensile membrane action stage later than S1 and S3-FR, because the unprotected secondary beams enhanced the slab capacity during the bending stage. It is noticeable that TMA was mobilised in each case at a deflection approximately equal to 0.95 of the slab thickness, irrespective of the presence of unprotected interior beams.

5.3.7 Failure modes

In S1, concrete cracking occurred around the column locations after about 12min of heating. However the test was continued, since there was no obvious indication of failure. After 86min of heating there was a loud sound caused by fracture of reinforcement along the interior side of the edge beams. Two full-depth cracks were observed: one close to a main beam and the other directly above a protected secondary beam (**Fig. 5.25**). Heating was stopped at this point, and no controlled cooling was conducted.

Fig. 5.25 Failure mode of S1

Test S2-FR-IB ended when fracture of reinforcement occurred along the interior side of the secondary edge beam and full-depth cracks appeared close to the edge beams, i.e. both main and secondary beams, as shown in **Fig. 5.26**.

Fig. 5.26 Failure mode of S2-FR-IB

In S3-FR, a compression ring formed after 28min of heating with the appearance of curved cracks at the four corners (**Fig. 5.27**). However, at 45min three full-depth cracks appeared suddenly, with one at the slab corner near the steel column, and two above the main beams. These cracks led to a ‘brittle’ failure of the compression ring and caused ‘run-away’ behaviour in S3-FR. The supporting compression ring could no longer be maintained, and therefore TMA was not able to develop further.

Fig. 5.27 Failure mode of S3-FR

In summary, the failure modes observed included:

- (1) Fracture of reinforcement along the interior side of the protected edge beams in all the tests;
- (2) Compression ring failure in S3-FR. No global collapse occurred in all the tests.

These failure modes differ in some respects from those considered in the SCI P288 design guide. In P288 the two common failure modes are fracture of reinforcement across the short span at the slab mid-span and compressive failure of the concrete at the slab corners. However, in the authors' tests no fracture of reinforcement at mid-span was observed, although cracks appeared at the slab centre. This is because the observed failure modes in these tests are the slab-beam collapse mechanisms, not only tensile membrane action mechanism in the slab. Comparisons between the test results and the predictions by the Bailey-BRE method were conducted to check if it is conservative when applied for the beam-slab systems (**Chapter 7**).

5.4 Numerical Simulations

In this section, the proposed model presented in **Chapter 3** was used to validate the test results of Series I, in terms of thermal and structural responses, and failure modes. The model was then used to provide insight into the stress distribution.

5.4.1 Proposed finite element model

As described in **Chapter 3**, the *Sequentially coupled thermal-stress analysis procedure* and the *Concrete damaged plasticity model* in ABAQUS/Explicit was used. Reinforcement was modelled using the layered rebar technique. Sequentially coupled thermal-stress analysis is performed by first solving the pure heat transfer problem, then reading the temperature solution into a stress analysis as a predefined field. In the stress analysis, the temperature can vary with time and position but is not changed by the stress analysis solution (Abaqus/CAE 2009).

A four-noded doubly-curved thin or thick shell element with reduced integration, the S4R shell element, was 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 full composite action between the steel downstand beams and the slab. An offset between the two tied surfaces was adopted to avoid any overlap between the two reference surfaces. This form of modelling assumed perfect bonding between the steel beam and the concrete slab and could not simulate accurately deterioration of full composite action between the steel downstand beams and the slab as temperature increases and as cracking took place. This is a shortcoming of the proposed numerical model.

Material properties of the steel and the concrete were obtained from tensile coupon tests and concrete cylinder tests conducted at ambient temperature. The material properties at elevated temperatures were then deduced using the material reduction factors specified in EN 1994-1-2 (2005d).

Fig. 5.28 Typical quarter FE model

Fig. 5.28 shows a simplified one-quarter numerical model (double symmetry), taking into account the protected and unprotected steel beams, the concrete slab, and the reinforcing mesh. Vertical support to the slab edges was provided by the protected edge beams. In turn, these beams were supported by the stocky columns. Thus, vertical restraint ($U3 = 0$) was imposed at the column connection locations; it was assumed that the vertical displacement at these positions was negligible. This

assumption was reasonable because the maximum recorded vertical displacement at the column support positions was only 3.1mm in S3-FR. Unfortunately, this value was not measured in S1 and S2-FR-IB. For S2-FR-IB and S3-FR, vertical restraint along the edge outstand ($U3 = 0$) was used to model the rotational restraint beam system; no springs were needed to model this beam system.

The recorded temperatures across the beam sections were entered directly into the model. For the slabs, the measured temperatures at the slab bottom surface were input, and then thermal gradient over the slab depth was ascertained through trial and error. Therefore, no heat transfer analysis was needed.

5.4.2 Model validations

5.4.2.1 Slab deflection and temperature development

Fig. 5.29 to **Fig. 5.31** presents comparisons of temperature distribution and slab deflections between the FE analyses and the experimental results. In these figures, the deflection caused by the load was not plotted, and the starting temperature was the ambient temperature. Thermal gradient over the slab thickness was determined by trial and error. With a thermal gradient of $10^{\circ}\text{C}/\text{mm}$ the predicted temperatures were very close to the recorded ones for S1 and S2-FR-IB. For S3-FR, even though the predicted temperatures at the slab top surface agreed very well with the measured ones, this was not the case for the mesh temperature in S3-FR (as seen in **Fig. 5.31(a)**). This was because severe cracks appeared above the protected edge beams in S3-FR rapidly leading to significant heat loss. As a result, the recorded mesh temperature increased at a slower rate after the cracks had appeared. The mesh temperature in S3-FR only increased from 91°C (after 22.5min of heating) after cracks appeared to 150°C (after 45min of heating).

As shown in **Figs. 5.29(b) - 5.31(b)**, the numerical predictions match well with the test results in terms of the slab deflection. However, there are still discrepancies between the two sets of results after the mesh temperature reached 200°C for S1, and 100°C for S3-FR. These are attributed to two reasons. Firstly, as the deflection increased, cracks in the concrete slabs developed, leading to significant heat loss.

However, the numerical model could not simulate this phenomenon, resulting in greater prediction of mesh temperature. Secondly, the numerical model could not predict concrete spalling, which had a significant effect on the slab behaviour. In S2-FR-IB the cracks were not as severe as those in S1 and S3-FR, and so the prediction for this test is considerably better.

Despite using a 2mm steel sheet, because of small pre-drilled holes in the sheet, small debris still fell into the furnace. After each test, the specimen was disposal and a lot of small concrete debris, like dust, was found. Also, separation of parts of concrete may occurred even though they did not drop. These led to the decrease of stiffness of the concrete slab, which the numerical model cannot capture.

Fig. 5.29 Comparisons between simulation and test for S1

Fig. 5.30 Comparisons between simulation and test for S2-FR-IB

a) Temperature distribution

b) Comparison of deflection

Fig. 5.31 Comparisons between simulation and test for S3-FR

5.4.2.2 Mid-span deflection of the edge beams against temperature

Temperatures of the beams at the beam bottom flanges which are shown in **Fig. 5.19** were input directly into the model, and thus no thermal analysis was needed. The results in terms of the mid-span deflections of the beams versus temperature are plotted from **Fig. 5.32** to **Fig. 5.34**.

Fig. 5.32 Comparisons of deflection of the edge beams – S1

Fig. 5.33 Comparisons of deflection of the edge beams – S2-FR-IB

Fig. 5.34 Comparisons of deflection of the edge beams – S3-FR

It should be noted that the outstand of S1 curled upwards freely. As deflection increased, towards the end of the test, this led to rotation of the beam top flange as can be seen in **Fig. 5.35**. Therefore, from 700°C onwards, the deflections of the edge beams decreased (**Fig. 5.32**). However, in the test, the slab outstand could not move upwards too much. This was because when the slab above the edge beams cracked, there was less rotation of the overhang compared to the numerical predicted rotations. Therefore, there is considerable disagreement towards the end of the test between the predicted and measured results. In S2-FR-IB and S3-FR, the slab outstand was restrained from upward movement, so the predicted results are in better agreement.

Fig. 5.35 Deformed shape of S1

For the main beam of S3-FR, although the trend was similar, the comparison is poorer because severe cracks appeared directly above the main beam top flange, leading to greater inaccuracy in the measurements. It should be noted that the beam deflections were measured from part of the slabs directly above the beams, not from the beam bottom flange because these beams were totally enclosed inside the furnace. In contrast, the part of the slab directly above the beams in S1 and S2-FR-IB, as well as that above the protected secondary beams in S3-FR, was fully composite with the beams. Therefore these recorded results are reliable.

5.4.3 Failure modes and stress distribution

Fig. 5.36 to **Fig. 5.38** show (a) stress distributions in reinforcement across the sections, (b) principal stress distribution in concrete at the top surfaces of the slabs at failure, and (c) principal maximum in-plane stress in concrete at the top surface of the slab. The failure times were 85.5min, 84min and 45min for S1, S2-FR-IB and S3-FR, respectively. In these figures, ‘Section 1’ denotes the mid-span section perpendicular to the protected secondary beams, ‘Section 2’ denotes the mid-span section perpendicular to the main beams, ‘OverMB’ is the cross-section above a main beam, and ‘OverPSB’ is the cross-section above a protected secondary beam. The positions of these sections are indicated in **Figs. 5.36(b)-5.38(b)**.

It can be seen that, for S1 at 85.6min (at mesh temperature of 391°C), the highest

tensile stress in the reinforcing mesh across a protected secondary edge beam is higher than that across the mid-span section of the slab (511MPa compared to 487MPa on Section 2). For S2-FR-IB at 84.0min (at mesh temperature of 512°C), the maximum tensile stress of 377MPa is located above the main beam; while the maximum stress in the reinforcement across the slab mid-span (Section 1) is 330MPa. It should be noted that reinforcing mesh in S1 and S2-FR-IB had yield and ultimate strengths of 543MPa and 771MPa, respectively. Therefore, on the basis of numerical simulations, in S1 and S2-FR-IB the fracture of reinforcement above the edge beams would occur first, before the fracture of reinforcement at the mid-span of the slabs. This observed failure mode in the model concurs with experimental observations.

a) Stress distributions in reinforcement across the sections

b) Principal stress distribution in concrete at the slab top surface and in the beams

c) Principal maximum in-plane stress in concrete at the top surface of the slab

Fig. 5.36 Numerical results of S1 at failure – 85.6 min

a) Stress distributions in reinforcement across the sections

b) Principal stress distribution in concrete at the slab top surface and in the beams

c) Principal maximum in-plane stress in concrete at the top surface of the slab

Fig. 5.37 Numerical results of S2-FR-IB at failure – 84.0 min

a) Stress distributions in reinforcement across the sections

b) Principal stress distribution in concrete at the slab top surface and in the beams

c) Principal maximum in-plane stress in concrete at the top surface of the slab

Fig. 5.38 Numerical results of S3-FR at failure – 45.0 min

For S3-FR, at failure of 45min (at mesh temperature of 150°C), the maximum tensile stress of 659MPa is found at the slab mid-span (Section 2), followed by that above the protected secondary beam (638MPa). In S3-FR, predicted failure is due to fracture of reinforcement at the slab mid-span. However, in the actual test, the failure mode was due to crushing of compression ring. As shown in **Table 5.4**, the equivalent plastic strains in uniaxial compression (PEEQ) at the slab top surfaces at its corners are 0.0057, 0.0031 and 0.0051, for S1, S2-FR-IB and S3-FR respectively. These values are approximately equal to the compressive strain $\varepsilon_{cu,\theta}$ (which corresponds to $f_{c,\theta}$) according to EN 1994-1-2, which would have been 0.0051, 0.0039 and 0.0033, for S1, S2-FR-IB and S3-FR, respectively at the prevailing temperature of the concrete slab. It can be observed that the equivalent plastic strains of S1 and S3-FR have exceeded the compressive strain $\varepsilon_{cu,\theta}$, which is allowable in the concrete plasticity model as shown in **Fig. 3.1**. This means that the failure has occurred at the slab corners for S1 and S3-FR, but not for S2-FR-IB. Unfortunately, there is no obvious indication of which failure mode occurred first, and it seems that fracture of the reinforcement is the most likely failure mode for the slabs which have interior beams.

Table 5.4 Strains at top surface of the slabs based on FE analyses – Series I

	<i>Failure time</i>	<i>Equivalent plastic strain in uniaxial compression at corners</i>	<i>Temperature at top surface (°C)</i>	<i>Compressive strain $\varepsilon_{cu,\theta}$ corresponding to $f_{c,\theta}$ in EN 1992-1-2</i>	<i>Maximum concrete strain $\varepsilon_{ce,\theta}$ in EC2 Pt.2</i>
<i>S1</i>	85.8	0.0057	172	0.0051	0.0243
<i>S2-FR-IB</i>	84.0	0.0031	95	0.0039	0.0223
<i>S3-FR</i>	45.0	0.0051	60	0.0033	0.0213

Therefore, it can be concluded that the model predicts reasonably well the failure modes of the specimens, including fracture of reinforcement above the edge beams and failure of compressive ring. For S1 and S2-FR-IB, these predicted failure modes concur with the experimental observations. However, for S1 and S3-FR, there is no obvious indication of which failure mode occurred first.

The principal stress distributions at the top surface of the slabs are shown in **Figs. 5.36(b)-5.38(b)**, in which negative values indicate compressive stresses and positive values tensile stresses. It can be seen that TMA was clearly mobilized in all the specimens, with the formation of a tensile zone at the slab centre and a peripheral ‘compression ring’. The compression ring consists of part of the concrete slab above the edge beams and part of the top flange of the edge beams.

5.5 Discussions

Results and observations from three one-quarter scale tests on composite slab-beam systems in fire (Series I) have been presented in **Section 5.3**. Two of the slabs (S1 and S3-FR) did not have interior composite beams; the other specimen (S2-FR-IB) had two interior beams. The purpose was to study the effect of interior beams on the system behaviour. The test setup was varied in order to investigate the effect of rotational edge-restraint on the slab behaviour.

In **Section 5.4**, the test results were used to validate the numerical model. The model is capable of predicting reasonably well the system behaviour in terms of temperature distribution, structural behaviour, and failure modes. This section provides discussions drawn out from the experimental observation and numerical studies.

5.5.1 Effect of interior beams

Fig. 5.39 shows a comparison of mid-span slab deflection against mesh temperature, while the test results are summarized in **Table 5.5**. For S1 and S3-FR, the yield load at the failure temperature of mesh reinforcement p_{y,θ_m} can be easily calculated on the basis of conventional yield-line theory. For S2-FR-IB, it is impossible to determine exactly when the interior beams made little contribution to the yield load. However, at failure of S2-FR-IB (84min), temperature of the unprotected interior beams already reached 892°C at which the reduction factor for yield strength of steel was only 0.064. Therefore, for the purpose of comparison it is reasonable to assume that at the failure of S2-FR-IB, the unprotected interior beams would provide no contribution to the slab capacity. In other words, the yield load of S2-FR-IB at the

mesh failure temperature was calculated on the basis of a structural configuration without the interior beams.

Fig. 5.39 Mid-span slab deflection vs. mesh temperature

Table 5.5 Summary of test results – Series I

	h_s mm	$p_{y,20}$ kN/m ²	Failure time min	Failure Temperature (°C)			p_{test} kN/m ²	p_{y,θ_m} kN/m ²	δ_{max} mm	δ_{max}/h_s	$p_{test}/p_{y,\theta_m}$
				θ_t	θ_m	θ_b					
S1	55	8.0	85.8	172	391	630	15.62	7.97	131	2.38	1.96
S2-FR-IB	55	36.8	84.0	95	512	664	15.13	5.93	177	3.22	2.55
S3-FR	58	10.4	45.0	60	150	351	15.98	10.35	115	1.98	1.54

S2-FR-IB failed at a deflection of 177mm when the mesh temperature had reached 512°C. The corresponding values were 131mm at 391°C and 115mm at 150°C, for S1 and S3-FR respectively. On the other hand, **Table 5.5** indicates that S2-FR-IB had the greatest enhancement factor, at 2.55 compared to 1.96 for S1 and 1.54 for S3-FR. This enhancement factor is defined as the ratio of the test failure load to the small-deflection yield-line failure load at the same mesh temperature. It is obvious that continuity of reinforcement over supporting beams, and the presence of interior beams, have significantly reduced the slab deflection and enhanced the slab's load-bearing capacity. The enhanced capacity is greatly contributed by the tensile forces in the interior beams, as observed in **Fig. 5.37(b)**.

The interior beams also considerably affect the magnitude of stress in the mesh reinforcement, as well as its distribution, and this may have led to differences in the failure modes of the floor assemblies. As can be seen in **Figs. 5.36(a)-5.38(a)**, in the case of S2-FR-IB the maximum tensile stress in the reinforcement across the mid-span of the slab was only 330MPa, which is smaller than the reinforcement stress above the edge beams. In contrast, the corresponding values were 487MPa and 659MPa, for S1 and S3-FR respectively; these are both greater than the reinforcement stress above the edge beams. Consequently, the failure mode may change from reinforcement fracture at mid-span (for specimens without interior beams) to reinforcement fracture in the vicinity of edge beams (for specimens with interior beams).

The difference in the stress distributions can be clearly identified in **Figs. 5.36(b)-5.38(b)**. For S2-FR-IB, the tensile stresses caused by TMA in the reinforcement above the interior beams are reduced, partly because of the superposed compressive stresses caused by sagging bending of the T-flange composite interior beams. For S1 and S3-FR with no interior beams, the tensile stresses are continuous with the maximum values at the centre of the slabs.

The effect of interior beams on the deflection of edge beams can be deduced from **Fig. 5.20**. At similar temperatures, with the same test setup, the protected secondary beams of S3-FR had a greater deflection than those in S2-FR-IB, because of the difference in load path from the slabs to the beams. During the initial heating stage, in S3-FR load was transferred directly to the protected edge beams, while in S2-FR-IB the load was transferred via unprotected interior beams to the protected edge beams. As temperatures increased, the load transfer mechanism of the two slabs became the same, since the unprotected interior beams progressively lost both stiffness and strength from 400°C onwards. Also, the main beam of S2-FR-IB had a slightly greater deflection than that of S1 because of the difference in load paths as described above.

5.5.2 Effect of rotational restraint

As shown in **Fig. 5.39**, when the mesh temperature was below 100°C S1 and S3-FR experienced similar deflections. This means that at small deflection (below 42mm), rotational continuity was maintained by the reinforcement over the supporting protected edge beams. Above this temperature, slab S3-FR experienced larger deflection than S1, due to the restraint beam system on top of slab S3-FR. This caused wide cracks to form over the main beams at a very early point, 20min after heating, while these cracks only appeared in S1 after 30min. This demonstrates that the presence of rotational edge-restraint became unbeneficial as the mesh temperature increased.

As can be seen in **Fig. 5.36(a)** and **Fig. 5.38(a)**, the maximum stresses in the reinforcement above the edge beams were significantly greater in S3-FR at 45min of heating than those in S1 at 85.6min of heating. The maximum stresses in the reinforcement over protected secondary and main beams in S3-FR were 638MPa and 614MPa, respectively, while the corresponding values in S1 were only 511MPa and 482MPa, respectively. It should be noted that the mesh temperatures of S1 after 85.8min of heating and that of S3-FR after 45min of heating were 391°C and 150°C, respectively. It means that the effect of temperature on reduction in the strength of reinforcement can be ignored. Therefore, the differences of stresses in the reinforcement in these two cases can be explained by the effect of rotational restraint on the stress distribution; where rotational restraint exists, the stresses above the edge beams are more intense. This results in severe concrete crushing at the four corners and wide tension cracks over the edge beams. Thus, the failure of the compression ring occurred quickly in S3-FR. Once the compression ring failed, the TMA mechanism could not be sustained, and consequently the slab failed.

Comparisons of beam deflection between S1 and S3-FR are shown in **Fig. 5.20**. It should be noted that the outstand of S1 were able to curl upwards freely, but in S3-FR this was not possible due to external restraint. It is observed that the rotational restraint has no effect on the vertical displacement of the main beams. On the other hand, deflection of the protected secondary beam of S3-FR was greater than those of

S1. This may be caused by the additional load of the slab outstand applied to the protected secondary beams of S3-FR.

5.6 Conclusions

This chapter presents experimental results and observations from three one-quarter scale composite beam-slab systems tested under fire conditions. The objectives were to study the effects of unprotected secondary beams and rotational edge restraint on structural behaviour. Additionally, FE models using ABAQUS/Explicit software have been developed to simulate the test results. On the basis of the experimental results and the numerical assessments, the following conclusions can be drawn:

- (1) The test setup was reliable because the sum of internal forces calculated from the data of strain gauges and the external applied load were in close agreement. The discrepancy is only 3.8%, 2.3% and 6% for S1, S2-FR-IB and S3-FR, respectively. On the other hand, the load was shown to distribute equally to four supporting columns during both the bending and tensile membrane stages. When cracks had developed significantly in the slab, part of the load was redistributed within the four columns. However, this load redistribution depended on the actual distribution of concrete cracks. It was difficult to predict a similar trend for all specimens.
- (2) Temperature distribution over the slab thickness depends mainly on the air temperature, and is independent of unprotected interior beams.
- (3) Reinforcement continuity over the supporting edge beams and the presence of interior beams can reduce deflection of the slab and greatly enhance its load-bearing capacity. The experimental enhancement factors were 1.96, 2.55, and 1.54 above the yield line capacity for S1, S2-FR-IB and S3-FR, respectively. It should be noted that S2-FR-IB had two unprotected interior beams, while S1 and S3-FR had no interior beams; and the yield line load at failure of each specimen was different since the specimens were failed at different temperatures.
- (4) The presence of interior beams affects significantly the magnitude and distribution of stresses in the mesh reinforcement. The maximum tensile

stress did not occur at the slab centre, but was located in the concrete slab above the protected edge beams. This may cause different failure modes for the floor assemblies, compared with the isolated slab panels.

- (5) Rotational restraint at the slab overhang induces intense stress concentration above the edge beams, which can result in premature concrete crushing at the four corners of the slab and wide cracks over the protected edge beams. The rotational restraint did not have any significant effect on the beams' vertical deflection.
- (6) The test specimens revealed different 'composite beam' failure modes of the floor assemblies. These include: (a) fracture of reinforcement in the vicinity of protected edge beams in all the tests; (b) in the specimens without unprotected interior beams, failure may occur through crushing of compression ring (S3-FR test). No fracture of reinforcement was observed at the mid-span of the slabs.
- (7) Irrespective of the presence of interior beams, tensile membrane action is mobilised at a deflection equal to approximately 0.9 to 1.0 of the slab thickness.
- (8) None of the connections had failed or fractured during the heating or the cooling phase.
- (9) The accuracy of a FE model in predicting the structural/thermal behaviour of composite beam-slab systems at large deflections is controlled by the accuracy of temperature predictions of mesh reinforcement.
- (10) A good correlation between the experimental and predicted results of temperature and deflection of the slab and the beams is obtained. Therefore, the proposed thermal/structural model can be used with confidence as a verified tool to predict the behaviour of composite beam-slab systems in fire.

CHAPTER 6 EXPERIMENTAL AND NUMERICAL STUDIES OF SERIES II

6.1 Introduction

As mentioned in **Chapter 2**, there is a lack of experimental studies on the effect of stiffness of protected edge beams on the behaviour of composite slab-beam systems in fire. There was only a numerical study conducted by Lim (2003), but without any physical tests. He conducted a parametric study to investigate the slab behaviour with different sizes of the supporting edge beams and used the span/20 deflection limit as the failure criterion. He concluded that as the edge beam size decreases, the slab fails at a lower load with a greater deflection.

Therefore, series II of this experimental programme focused on the investigation of the effect of bending stiffness of the fire-protected edge beams (main and secondary beams) on the behaviour of CB-S systems in fire. This chapter is organized as follows. In **Section 6.2**, the specimens, the investigated parameters and instrumentation are described. **Section 6.3** discusses the performance and experimental observations of the specimens. In **Section 6.4**, the proposed numerical model is used to simulate and to provide deeper insights into the test results. **Section 6.5** highlights the effect of the investigated parameters on the performance of the CB-S systems. Conclusions are provided in **Section 6.6**.

The *objectives* of Series II are to:

- (1) investigate the effect of bending stiffness of *the protected secondary edge beam* on the fire behaviour of the CB-S systems;
- (2) study the effect of bending stiffness of *the protected main beam* on the fire behaviour of the CB-S systems;
- (3) capture the failure modes and deflection shapes of the systems;
- (4) provide experimental results to validate the proposed numerical model.

6.2 Test Specimens

The numerical analyses presented in **Section 3.5.7** show that in terms of the four properties of the edge beams (steel grade, torsional rigidity GI_t , bending stiffness about the major axis EI_y , bending stiffness about the minor axis EI_z), only the bending stiffness about the major axis EI_y has significant effect on the membrane behaviour of floor assemblies. Therefore, the bending stiffness about the major axis EI_y was chosen as the main parameter in Series II.

Series II consisted of five specimens in total, denoted as P215-M1099, P368-M1099, P486-M1099, P215-M1356, and P215-M2110. In this nomenclature, for example, P215-M1099 indicates a specimen which has the second moment of area about the major axis of protected secondary edge beam (I_{yPSB}) as 215cm^4 , and that of main edge beam (I_{yMB}) as 1099cm^4 . It should be noted that the bending stiffness (EI_y) of a section increases linearly to the second moment area of that section (I_y). Therefore, the term ‘stiffness’ also refers to ‘second moment of area’. **Table 6.1** shows the specimens with different sections of the edge beams. All specimens had two unprotected interior beams which were fabricated I-sections of 80x80x17.3. The dimensions 80x80x17.3 refer to a section of 80mm depth, 80mm width, and 17.3kg/m mass.

Table 6.1 Specimens of Series II

Specimens	MB		PSB		Relative Ratio	
	Section	$I_{yMB} (\text{cm}^4)$	Section	$I_{yPSB} (\text{cm}^4)$	MB	PSB
P215-M1099	W130x130x28.1	1099	80x80x17.3	215		
P368-M1099	W130x130x28.1	1099	100x80x18.8	368		1.71
P486-M1099	W130x130x28.1	1099	Joists 102x102x23	486		2.26
P215-M1356	UB 178x102x19	1356	80x80x17.3	215	1.23	
P215-M2110	UB 203x102x23	2110	80x80x17.3	215	1.92	

P215-M1099 was chosen as the control specimen with its relative stiffness as unity (**Table 6.1**). To investigate the effect of bending stiffness of secondary edge beam, the values EI_{yPSB} of P368-M1099 and P486-M1099 were increased to 1.71 and 2.26

times, respectively, compared to that of control specimen P215-M1099. The main and unprotected interior secondary beams for these specimens were kept the same.

Similarly, to study the effect of bending stiffness of main beam, the values EI_{yMB} of P215-M1356 and P215-M2110 were increased to 1.23 and 1.92 times, respectively, compared to that of control specimen P215-M1099. The secondary edge beams and unprotected interior beams of these specimens were kept the same.

Fig. 6.1 shows a typical specimen in Series II. The slabs were 2.25m long and 2.25m wide with an outstand of 0.45m around the four edges. Five M24 black bolts were cast into the edge of the concrete slab. Half of these bolt lengths were cast together with the concrete slabs, while the other half were attached to the in-plane restraint system as shown in **Section 4.4.4**. The locations of these bolts were secured by using 8mm thick steel plate along the concrete slab edges. The purpose of these bolts was to simulate the continuity of interior slab panels. These bolts applied rotational restraint but allowed horizontal movement in straight lines along the slab edges as explained in **Section 4.3.2.2**.

Fig. 6.1 Typical specimen in Series II

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

6.2.1 Material properties

Material samples were tested at ambient temperature to obtain the stress-strain curves. The material properties at elevated temperatures were assumed to vary according to EN 1994-1-2 (2005d) in which the reduction factors for strength and stiffness of concrete and steel are well established (**Section 2.2**).

Reinforced concrete slabs

The specimens were cast using ready-mixed concrete, with the aggregate size ranging from 5 to 10mm. P215-M1099 was cast together with S3-FR (series I). P368-M1099 and P215-M1356 were cast in the same batch, while P486-M1099 and P215-M2110 were cast in another batch. For each batch, six cylinders were tested at 28 days to determine the mean compressive strength f_{cm} . The mean tensile strength f_{ctm} and the mean elastic modulus E_{cm} were calculated in accordance with EN 1992-1-1 (2004b). **Table 6.2** summarises the properties of the concrete slabs, such as the slab thickness h_s , effective depths d_x , d_y , concrete cylinder strength f_{cm} , etc.

Table 6.2 Properties of concrete slabs – Series II

	h_s mm	d_x mm	d_y mm	f_{cm} MPa	f_{ctm} MPa	E_{cm} GPa	f_y / f_u MPa	Ultimate strain %	Modulus E_s , GPa
<i>P215-M1099</i>	57	33	30	31.3	2.4	31.0	689/806	14.8	203.4
<i>P368-M1099</i>	58	32	29	32.9	2.6	31.4	689/806	14.8	203.4
<i>P486-M1099</i>	55	33	30	28.9	2.3	30.3	689/806	14.8	203.4
<i>P215-M1356</i>	58	34	31	32.9	2.6	31.4	689/806	14.8	203.4
<i>P215-M2110</i>	59	42	39	28.9	2.3	30.3	689/806	14.8	203.4

Shrinkage reinforcement mesh with a grid size of 80mm x 80mm and a diameter of 3mm (giving a reinforcement ratio of 0.16%) was placed about the mid-height of the concrete slabs. The mesh was continuous across the whole slab, with no lapping of mesh. Four samples were tested to determine the yield and ultimate strengths of the

mesh. All specimens used a similar cold-worked steel mesh with a yield strength of 689MPa and an ultimate strength of 806MPa. Elongation of the mesh was 14.8%, and the elastic modulus was 203.4GPa. Since 3mm diameter rebars were used, it was impossible to mount strain gauges on them. Therefore, the elastic modulus was determined based on the extensometer readings. Also, due to cold-worked steel the stress-strain curve did not show a clearly defined yield point. Thus, the 0.2% proof stress was taken as the yield strength. **Fig. 6.2** shows the stress-strain relationship of reinforcement used.

Fig. 6.2 Stress-strain relationship of reinforcement – Series II

There was no standard steel decking suitable for the slabs due to 1:4 scaling. To protect the heating elements from concrete spalling, the slabs were cast onto a 2mm thick steel sheet with small pre-drilled holes. The contribution of this decking to the slab's load-bearing capacity could be ignored, since the unprotected sheet would debond from the concrete slab.

Structural steels

Tensile coupon tests, two from the web and two from the flanges for each type of I-section used (**Table 6.1**), were conducted at ambient temperature. For each coupon, two sources of reading were obtained, *viz.* two strain gauges mounted onto both sides of the coupon and an extensometer. **Fig. 6.3(a)** indicates that the two results were rather consistent. Thus, they can be used to determine the elastic modulus of the steel

section. **Table 6.3** summarises the average results from tensile tests and the measured geometrical properties of the beams.

Fig. 6.3 Typical stress-strain curve of structural steel for Series II

Table 6.3 Details of steel beams – Series II

Specimens	Denote	Depth	Width	Thickness (mm)		Yield strength	Ultimate strength	Elastic modulus
		h_{sb} (mm)	b_f (mm)	Web	Flange	f_y (MPa)	f_u (MPa)	E_s (GPa)
P215-M1099	MB1	131	128	7.0	11.0	307	462	211.4
	PSB1	80	80	10.3	10.0	467	588	210.6
	USB	80	80	10.3	10.0	467	588	210.6
P368-M1099	MB1	131	128	6.8	11.0	307	462	211.4
	PSB2	100	80	10.3	10.0	467	588	210.6
	USB	80	80	10.3	10.0	467	588	210.6
P486-M1099	MB1	131	128	7.0	11.0	307	462	211.4
	PSB3	102	101	7.2	8.7	356	510	205.4
	USB	80	80	10.3	10.0	467	588	210.6
P215-M1356	MB2	178	101	5.1	8.2	374	500	202.5
	PSB1	80	80	10.3	10.0	467	588	210.6
	USB	80	80	10.3	10.0	467	588	210.6
P215-M2110	MB3	203	102	5.4	9.7	425	561	205.4
	PSB1	80	80	10.3	10.0	467	588	210.6
	USB	80	80	10.3	10.0	467	588	210.6

6.2.2 Instrumentation

Fig. 6.4 Arrangement of thermocouples and LVDTs

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 to five specimens was used as shown in **Fig. 6.4**. Temperature of the slab was measured at Sections 1, 2 and 3 (**Fig. 6.4(a)**). At each section, temperature was monitored at the top and bottom surfaces, and at the reinforcing mesh. Temperatures of the beams were measured at Sections A to F at the bottom and top flanges, and at the middle of the beam web (**Fig. 6.4(a)**). The furnace air temperature was also monitored.

A total of 20 LVDTs were used to measure displacements of the floor assemblies as shown in **Fig. 6.4(b)**. L1 to L3 consisted of 300mm LVDTs to monitor vertical deflection of the slab. L4 to L11 comprised 200mm LVDTs to measure vertical deflection of the beams. Three 50mm LVDTs (L12, L13 and L14) were used to measure horizontal and vertical displacements of the column. L15 to L20 were the 100mm LVDTs to record horizontal displacement along the slab edges.

6.2.3 Initial applied load

Load was applied to a predetermined value, and was kept constant during the heating stage. After failure had been identified, the load was removed and the cooling stage took place naturally.

The magnitude of the total applied load for all specimens was 15.8kN/m^2 . This value was equal to 0.35 times the conventional yield-line load at ambient temperature, which was 45.1kN/m^2 calculated based on the slab configuration with two interior beams using the principle shown in **Section 4.3.3**. The strength of supporting beams does not affect the yield-line load. The selection of load ratio is within the lower practical range of 0.3 to 0.7.

6.3 Test Results and Observations

In Series II, the criteria for ‘failure’ were similar to Series I, which corresponded to the time when either one of the conditions took place: (1) Full-depth cracks with the maximum crack width of about 10mm in the vicinity of the edge beams or failure of compression ring near the column region; or (2) A significant drop in the mechanical resistance, and the load could not be maintained.

To present clearly the results, five specimens were divided into two groups. *Group 1* specimens consisting of P215-M1099, P368-M1099, and P486-M1099 is to study the effect of stiffness of the *protected secondary edge beams*; *Group 2* comprising P215-M1099, P215-M1356, and P215-M2110 is to investigate the effect of stiffness of the *protected main beams*.

6.3.1 Temperature distributions in the slabs

6.3.1.1 Temperature distributions in horizontal directions

Temperatures of the slabs were measured at their bottom and top surfaces, and at the reinforcing mesh at Sections 1, 2 and 3 as shown in **Fig. 6.4(a)**. These temperatures were compared to check if there was double symmetry in the temperature distribution of the slabs in the x-x and y-y directions. The temperature measurements

of all specimens confirmed that temperature only varies over the slab thickness, and there was double symmetry. **Figs. 6.5-6.6** show the results of P215-M1099 and P215-M1356. In these figures, SB, S, and ST stand for thermocouples at the bottom surface, the reinforcing mesh and the top surface of the slabs, respectively.

Fig. 6.5 Temperature distribution in horizontal directions of P215-M1099

Fig. 6.6 Temperature distribution in horizontal directions of P215-M1356

6.3.1.2 Temperature distributions across the slab thickness

Fig. 6.7 shows the temperature distribution across the slab thickness against time of the specimens. Temperatures were set by the furnace controller in two steps: (1) increase from room temperature up to 1000°C within 60min, and (2) keep the furnace temperature at 1000°C during the next two hours. From trial tests without specimens (shown in **Fig. 4.2**), this was the maximum heating rate which can be

created by available power capacity in the laboratory. The slabs were estimated to have failed within two hours under fire conditions; therefore a period of two hours was chosen. When there was a specimen above the furnace, air temperatures inside the furnace could not follow the fire curve set by the controller, because the gaps could not be completely filled as the slab deflected. As can be seen in **Fig. 6.7**, up to 40min of heating, when there were a few concrete cracks, the heating rate was about $15^{\circ}\text{C}/\text{min}$. From 40min onwards, as the concrete cracks developed greater, heat loss was generated, leading to a much lower heating rate of about $6^{\circ}\text{C}/\text{min}$.

a) P215-M1099

b) P368-M1099

c) P486-M1099

d) P215-M1356

e) P215-M2110

Fig. 6.7 Temperature distributions and deflection vs. time – Series II

It can be seen in **Fig. 6.7** that up to 40min of heating, temperature of the slabs increased slowly due to inherent moisture content in the concrete. As the temperature increased, the moisture content reduced leading to a greater heating rate. The maximum temperature at the bottom surface was 824°C, 854°C, 760°C and 835°C in P215-M1099, P368-M1099, P486-M1099 and P215-M1356, respectively.

The temperature development in P215-M2110 is shown in **Fig. 6.7(e)**. Its heating rate of 13.1°C/min up to 25min was close to the heating rate of the other tests. However, air temperature increased at a very low rate right after 30min of heating. From 30min to 198min, the air temperature increased from 350°C to only 820°C, giving a very slow heating rate of only 2.7°C/min. Investigation after this test showed that a heating element was badly damaged, caused by concrete spalling. Therefore, temperatures of P215-M2110 increased at a very slow rate. At failure of 198min, the maximum temperature at the bottom surface, the reinforcing mesh level and the top surface were 715°C, 508°C and 228°C, respectively. In spite of the slow heating rate, the test results could be used to verify the FE models, and then numerical investigation with the same parameters of P215-M2110 could be conducted.

Temperature at the slab top surface was much lower than the air temperature due to the concrete heat sink effect. For example, the maximum value was only 228°C in P215-M2110, compared to its maximum air temperature of 820°C.

6.3.2 Slab displacements

6.3.2.1 Mid-span deflection

Fig. 6.7 shows the deflection of the slabs against time. A similar trend can be observed. Up to 40min of heating, the slabs deflected at a lower rate due to a small thermal gradient generated over the slab thickness. As the temperature increased, the effect of thermal bowing was more obvious, leading to a greater rate of deflection. All specimens failed due to occurrences of full-depth cracks in the vicinity of the protected edge beams. **Table 6.4** summarises the failure times of the specimens.

Table 6.4 Failure times of specimens – Series II

Specimen	h_s <i>mm</i>	Failure time <i>min</i>	Temperature at failure (°C)			Maximum deflection w_m <i>mm</i>	w_m / h_s
			θ_{top}	θ_{mesh}	θ_{bottom}		
<i>P215-M1099</i>	57	72.2	108	348	602	124	2.17
<i>P368-M1099</i>	58	74.6	118	316	688	118	2.03
<i>P486-M1099</i>	55	98.2	127	412	697	139	2.52
<i>P215-M1356</i>	58	76.3	101	351	734	121	2.09
<i>P215-M2110</i>	59	198.0	228	508	715	143	2.42

Table 6.4 indicates that all the slabs experienced large deflections, with the maximum deflection of 143mm in P215-M2110 which equals to $2.42h_s$ (h_s is the slab thickness). Referring to the heating of unprotected interior beams (**Section 6.3.3.1**), at the failure time of all specimens, it can be found that these beams were already heated to about 700°C. Apparently, at this level of heating, the interior beams would no longer be able to resist the applied load. As a result, TMA was progressively activated in the concrete slabs. The mobilisation of TMA can be clearly illustrated through the development of lateral displacement along the slab edges (**Section 6.3.2.2**).

P215-M2110 failed last, at 198min of heating. Unfortunately, it could not be concluded that this was due to the greatest stiffness of the main beams. This could be due to a very low heating rate of 2.7°C/min, caused by the damage of a heating element in the furnace.

6.3.2.2 Horizontal displacement of the slabs

Figs. 6.8 to 6.10 show the horizontal displacements of P215-M1099, P368-M1099 and P486-M1099, respectively. In these figures, positive value indicates inward horizontal displacement, and negative value outward displacement. The positions of the LVDTs are shown in **Fig. 6.4**. L15, L16, L17 were placed along the edge parallel to the main beam, while L18, L19, L20 were placed parallel to the secondary edge beam. Three important observations can be drawn from the tests.

Fig. 6.8 Horizontal displacement of the slab edges vs. time – P215-M1099

Fig. 6.9 Horizontal displacement of the slab edges vs. time – P368-M1099

Fig. 6.10 Horizontal displacement of the slab edges vs. time – P486-M1099

Firstly, the horizontal displacement-time relationship of the slabs can be divided into three stages. *In the first stage*, the slab edges moved outwards due to thermal expansion as the temperature of the slab increased. Up to 20min of heating, the temperature was low because the moisture in the slab had gradually released. Therefore, displacements of the edges in two horizontal directions were small, only about 2mm. From 20min to about 50min of heating, the displacements increased at a greater rate with the maximum values of 8mm, 9.7mm and 9mm for P215-M1099, P368-M1099 and P486-M1099, respectively. *In the second stage*, after 50min, when the slab deflection reached about $1h_s$, tensile membrane action was mobilised. Therefore, the sum of outward displacements due to thermal expansion and inward displacement due to TMA resulted in an almost constant displacement. *In the third stage*, when the slab experienced large vertical deflections, the slab edges moved inwards significantly. However, intense tensile stresses in the reinforcement above the protected edge beams led to failure in these regions. When the 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 displacement rate reduced and was almost constant. As explained above, this resulted from the sum of outward displacement due to thermal expansion and inward displacement due to TMA. 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 6.3.3.2**), and the development of crack patterns in the slabs (**Section 6.3.4**).

Thirdly, it can be observed that the recorded horizontal displacements from L15, L16 and L17 along one slab edge and those from L18, L19 and L20 along the transverse slab edge were only slightly different. This indicates that the slab edges initially moved outwards and then inwards in straight lines. Therefore, it can be concluded that the tested slab panels could accurately simulate the continuity conditions of interior panels.

The horizontal displacements of P215-M1356 and P215-M2110 which are plotted in **Fig. 6.11** and **Fig. 6.12** respectively, showed similar observations with the first three tests. TMA was mobilised at 56.2min and 125min for P215-M1356 and P215-M2110, respectively. This was because at these times, the sum of outward displacements due to thermal expansion and inward displacements due to TMA resulted in a constant displacement rate. On the other hand, the displacements along the slab edges were also slightly different. This indicated that the slab edges moved initially outwards and then inwards in straight lines.

Fig. 6.11 Horizontal displacement of the slab edges vs. time – P215-M1356

Fig. 6.12 Horizontal displacement of the slab edges vs. time – P215-M2110

Table 6.5 summarises the times at the onset of TMA which are determined based on the horizontal displacement-time curve of the slabs. The corresponding vertical slab deflections are also indicated. As can be seen, TMA was mobilised at a slab deflection of 0.86 to 1.0 of the slab thickness, irrespective of the stiffness of the edge beams.

Table 6.5 Time when TMA mobilised – Series II

<i>Specimen</i>	<i>Thickness h_s (mm)</i>	<i>Failure time (min)</i>	<i>TMA activated (min)</i>	<i>Vertical deflection, w, when TMA activated (mm)</i>	<i>w_{TMA}/h_s</i>
<i>P215-M1099</i>	57	72.2	51	58.6	1.03
<i>P368-M1099</i>	58	74.6	54	53.9	0.93
<i>P486-M1099</i>	55	98.2	52	53.7	0.98
<i>P215-M1356</i>	58	76.3	56	49.7	0.86
<i>P215-M2110</i>	59	198.0	127	51.1	0.87

6.3.3 Behaviour of steel frames

6.3.3.1 Temperature developments of the beams

Fig. 6.13 shows the temperature evolution at the bottom flange and mid-span deflection of the beams against time from the onset of heating, *viz.* protected main beam (MB), protected secondary edge beam (PSB), and unprotected interior beam (USB).

In the first three tests (P215-M1099, P368-M1099, and P486-M1099), temperature at the bottom flange of MB, PSB and USB was almost similar. This was because the beam depth was only slightly different, at 131mm, 100mm and 80mm for MB, PSB and USB, respectively. The effect of fire protection for these beams can be seen via their deflection-temperature relationship. The protected main and secondary edge beams deflected gradually, while the unprotected interior beams deflected at much greater rates.

In P215-M1356 and P215-M2110, temperature at the bottom flange of MB was greater than those of PSB and USB. This was because the depth of the main beam was 178mm and 203mm for P215-M1356 and P215-M2110 respectively (see **Table 6.3**), which were greater than the depth of PSB and USB at 80mm.

a) P215-M1099

b) P368-M1099

c) P486-M1099

d) P215-M1356

e) P215-M2110

Fig. 6.13 Deflections and temperatures of the beams vs. time – Series II

6.3.3.2 Mid-span deflections of the beams

Fig. 6.13 plots the mid-span deflection of the beams. A similar behaviour of the protected main and secondary edge beams was observed. Up to 60min of heating, these beams deflected at a small rate since they were fire-protected. From 60min onwards, the beams gradually lost fire-proofing and deflected at a significantly greater rate.

On the other hand, the mid-span deflection of the unprotected interior beam (measured by L3) followed closely the slab deflection in all the tests (measured by L1 and L2). This indicated that the slabs acted as a membrane under fire conditions.

6.3.3.3 Supporting columns

Horizontal displacements

Figs. 6.14-6.15 show the horizontal displacements of the supporting columns measured by L12 and L13. In these figures, positive and negative values indicate outward and inward displacements of the column from the slab centre, respectively. L12 was used to measure the horizontal displacement of the column along the secondary edge beam, while L13 was placed along the main beam direction (**Fig. 6.4**).

A similar trend in the displacement-time curve can be observed. The first stage began with outward displacements in both directions of the column due to thermal expansion of the edge beams as temperature increased. The maximum value was 5mm for L12 of P215-M1099 in Group 1 and 8mm for L13 of P215-M1356 in Group 2. In the second stage, when the slab deflected greatly the column moved back to its original position and then underwent significant inward displacements caused by pulling-in of the edge beams.

The turning point between outward and inward displacements can be considered as an indication of the onset of TMA, which appeared after about 50min of heating. It is noteworthy that the time coincided with that based on the horizontal displacement of the slab edges (**Section 6.3.2.2**).

In P215-M2110, although the trend was similar, the time at which TMA was mobilised was delayed, at 127min after heating. This was because the heating rate was very small ($2.7^{\circ}\text{C}/\text{min}$) compared to all the other tests (about $5.5^{\circ}\text{C}/\text{min}$) due to the damage of a heating element inside the furnace. P15-M2110 failed at 198min due to reinforcement fracture above the edge beams. At this time, the column inward displacement was still small. This was possibly resulted from the mesh temperature, which already reached 502°C , leading to an early failure at tensile membrane stage.

Fig. 6.14 Horizontal displacements of columns vs. time – Group 1

Fig. 6.15 Horizontal displacements of columns vs. time – Group 2

Vertical displacement

Vertical displacements of the columns are plotted in **Fig. 6.16** and **Fig. 6.17** for Group 1 and Group 2 specimens, respectively. Positive and negative values indicate downward and upward displacements, respectively.

Fig. 6.16 Vertical displacement of columns vs. time – Group 1

Fig. 6.17 Vertical displacement of columns vs. time – Group 2

It can be observed that all the specimens experienced a similar behaviour. Initially, the columns deflected axially downwards caused by the applied load of 15.8kN/m^2 . This UDL which corresponded to 0.35 times the yield load at ambient temperature only resulted in a small vertical displacement. The maximum column axial deformation was only 1.7mm in P215-M1356. When heating started, the column moved upwards due to thermal expansion, and then returned to its original position

due to pulling-in of the edge beams. For instance, the column in P215-M1099 moved back to its original position after 23.7min. As temperature increased, the column elongation increased because there was no axial restraint from thermal expansion. In Group 1 sub-structures, the maximum thermal elongation of the columns was 7mm for P368-M1099 and P486-M1099; in Group 2, the maximum value was 9.3mm in P215-M2110.

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. The onset of TMA marked by this indication is very consistent compared with the other two indications, i.e. horizontal displacement of the slab edges and horizontal displacements of the columns. The times corresponded to the turning points of the columns are shown in **Figs. 6.16-6.17**.

Deformed shapes of the columns

The deformed shapes of the protected steel columns after cooling are shown in **Fig. 6.18**. The columns were subjected to biaxial bending due to pulling-in of the edge beams in two directions. The columns were shown to be designed adequately as there was no premature failure.

a) P215-M1099

b) P368-M1099

c) P486-M1099

d) P215-M1356

e) P215-M2110

Fig. 6.18 Specimens after cooling – Series II

6.3.3.4 Connections

As shown in **Fig. 6.18**, none of the connections failed or fractured in any of the substructure tests, either during the heating or the cooling phase. This indicates that, where there is limited axial restraint, if the connections are designed in accordance with EN 1993-1-8 (2005c), and are protected in fire to the same rating as the connected members, the composite slab can mobilise TMA without connection failure during the heating or the cooling phase.

6.3.4 Development of crack patterns

The development of crack patterns was observed carefully during the tests and is re-plotted in **Fig. 6.19**. The heating times when cracks occurred are indicated, together with the corresponding temperatures in the reinforcing mesh and the corresponding mid-span slab deflections. The crack sequences shown by the numbers were consistent in all the tests.

a) P215-M1099

b) P368-M1099

c) P486-M1099

d) P215-M1356

* compression ring formed; ** failure occurred

e) P215- M2110

Fig. 6.19 Development of crack pattern – Series II

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, parts of the outstand were in biaxial bending but restrained by the columns. Therefore, these cracks consistently formed at an angle ranging from 30° to 45° to the slab edges.

Secondly, cracks appeared in the vicinity of the main beams (crack 2). The diagonal cracks emanating from the slab corners to the columns (crack 3) were then observed. These diagonal cracks (3) were caused by the bolts which restrained the slab. As the temperature increased, cracks along the protected secondary edge beams (crack 4) appeared. In P368-M1099, P215-M1356 and P215-M2110, cracks above the unprotected secondary beams were also observed (crack 4a). These cracks were probably formed due to compatibility of deflections between the slab and the unprotected interior steel beam. However, these cracks seemed to close up after cooling due to elastic rebound of the slabs.

In Group 1, in P215-M1099 test, after 51 minutes (min) of heating, a compression ring began to form when the mesh temperature had reached 181°C at a deflection of 59mm, $1.03h_s$. In P368-M1099 and P486-M1099, a compression ring began to form at the respective mesh temperature of 151°C and of 185°C, after 54min and 52min of heating. The corresponding deflections were 54mm and 54mm, equal to $0.93h_s$ and

$0.98h_s$, respectively. In Group 2, in P215-M1356, a compression ring formed at the mesh temperature of 198°C after 56min of heating, corresponding to a slab deflection of 50mm ($0.86h_s$). In P215-M2110, due to a very low heating rate ($2.7^{\circ}\text{C}/\text{min}$), a compression ring formed last, at 127min after heating and at the mesh temperature of 343°C . The corresponding deflection was 51mm, or $0.87h_s$.

Based on the aforementioned observations, it can be concluded that TMA was mobilised at a slab deflection equal to 0.9 to 1.0 of the slab thickness h_s , irrespective of the stiffness of the protected edge beams. Tensile membrane mechanism appeared in these tests consisted of radial tension in the central area of the slab and a peripheral compression ring. Due to its self-equilibrating nature, horizontal edge restraint was not required for the mobilisation of TMA.

The onset of TMA can be recognised by appearances of the diagonal cracks inside the slab panel at four corners (crack 5). The more obvious indications were the transitions between outward and inward displacements of the slab edges, as well as of the columns (**Sections 6.3.2.2 and 6.3.3.2**). These three indications coincided very well in terms of time.

Severe cracks also appeared at the slab outstand (crack 6) as observed in **Fig. 6.19**. 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 the cracks near the end of the bolts. When the slab experienced very large deflections, the cracks opened through the slab thickness and reinforcement was fractured. However, this did not affect the test results because it happened after the failure had been identified.

6.3.5 Failure modes

The failure mode observed in Series II was fracture of reinforcement close to the protected edge beams (at crack 2 or 4). No run-away failure was observed. Shear studs and connections were shown to be designed adequately. No premature failure at the shear studs or at the connections was observed. **Figs. 6.20 to 6.24** show the

final failure modes of the specimens after cooling, in which the crack positions were indicated.

Fig. 6.20 Failure mode of P215-M1099

Fig. 6.21 Failure mode of P368-M1099

Fig. 6.22 Failure mode of P486-M1099

Fig. 6.23 Failure mode of P215-M1356

Fig. 6.24 Failure mode of P215-M2110

6.4 Numerical simulation

6.4.1 Proposed finite element model

The test results of Series II are used to validate the numerical model. Once validated, the numerical model can provide further information which could not be measured during the tests under fire conditions, e.g. stress-strain of reinforcement.

Solution strategy and input parameters were described in detail in **Sections 3.2** and **3.3**. It is noteworthy to remember that the *sequentially coupled thermal-stress analysis procedure* in ABAQUS/Explicit was adopted. Therefore, the thermal/structural behaviour of the composite floor assemblies can be obtained by using recorded temperature data without conducting thermal analysis.

The numerical model takes into account of the steel beams, the concrete slab, and the reinforcing mesh as shown in **Fig. 6.25**. The edge and interior beams were supported by the columns. The vertical displacements of the columns were assumed to be negligible. This assumption is reasonable because the maximum recorded column vertical displacement was only 9mm in P215-M2110 as shown in **Fig. 6.17**, about 6% of the maximum slab deflection. Besides, these columns did not show any imminent sign of failure. Therefore, vertical restraint ($U_3 = 0$) was imposed at the column locations.

Fig. 6.25 Typical proposed FE model

Vertical restraint along the edge outstand was used to model the rotational restraint beam system; 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, the first with fully horizontal restraint along the slab edges (*Case 1*), the second without any horizontal restraint (*Case 2*).

Concrete damaged plasticity model was used to simulate the concrete slab and *layered rebar technique* was adopted to simulate mesh reinforcement in the slab. Coupon tests and concrete cylinder tests were conducted at ambient temperature to determine the material properties of the steel and the concrete (**Section 6.2.1**). The stress-strain relationships and thermal properties of the materials at high temperatures were deduced according to EN 1994-1-2 (2005d).

A four-noded doubly-curved thin or thick shell element with reduced integration scheme (S4R shell element) was used to model 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 interaction* 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 reference surfaces. This form of modelling assumed perfect bonding between the steel beam and the concrete slab.

6.4.2 Thermal response

Using the recorded temperatures, temperature development across the slab thickness could be identified without any heat transfer analysis needed. However, since S4R shell elements were used to discretize the slabs, it was not possible to assign all the recorded temperatures at the slab bottom surface, reinforcing mesh and top surface into the model. Therefore, only the temperature at the slab bottom surface was input directly into the model. Thermal gradient of the slab was determined through trial and error to yield temperatures on the slab top surface as close as possible with the measured values. In order to validate the structural response, the predicted

temperatures from the analyses were first compared with the measured temperatures from the tests as shown in **Fig. 6.26**.

Fig. 6.26 Comparison of predicted and measured temperatures of the slabs - Series II

For the first four specimens, with a thermal gradient of $12^{\circ}\text{C}/\text{mm}$, 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. For P215-M2110, unexpected damage of one heating element resulted in a lower heating rate. Therefore, by trial and error, a thermal gradient of $8.5^{\circ}\text{C}/\text{mm}$ gave the best prediction for P215-M2110.

The S4R shell elements were also used to model the beams. The recorded temperatures at the top and bottom flanges and at the beam web can be input directly into the model without any thermal analysis. As a result, the recorded and predicted temperatures are consistent, and no comparison is needed.

6.4.3 Structural response

6.4.3.1 Vertical deflection of the slabs

Comparisons between the predicted and measured deflections of the slabs and the unprotected interior beams are plotted in **Fig. 6.27**. For the slab, the deflection is plotted versus the mesh temperature, whereas for the beam, it is 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 of the slab edges were analysed. In *Case 1*, the slab edges were fully restrained horizontally while in *Case 2*, the slab edges did not have any horizontal restraint.

a) P215-M1099

b) P368-M1099

c) P486-M1099

d) P215-M1356

Fig. 6.27 Comparison of predicted and measured deflections of the unprotected interior beam and the slab – Series II

Theoretically, the measured mid-span slab deflection should lie in between the predictions by *Case 1* and *Case 2* because the actual inplane restraint system should provide 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 was increased from 150°C to 250°C, the slab had greater deflections than the predictions. This may be caused by the cracks appearing above the edge beams, which led to a greater rotation of the slabs about the edge beams compared to that predicted in the simulations.

For P215-M1099 and P368-M1099, there is a gap between the test result and the prediction when the mesh temperature increased from 150°C to 250°C. This discrepancy can be explained by disagreement of the measured and predicted temperatures in reinforcing mesh as shown in **Fig. 6.26(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 controlled by the accuracy of temperatures at reinforcing mesh of the slab. Generally, the model still predicts relatively well the slab behaviour.

Comparisons between the predicted and the measured mid-span deflection of the interior beams for the two cases, *viz.* with and without inplane horizontal restraint, are also shown in **Fig. 6.27**. It can be seen that a good correlation between the two results was obtained.

6.4.3.2 Vertical 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 (fully horizontal restraint along the slab edges) and case 2 (no horizontal restraint along the slab edges) are plotted in **Fig. 6.28**. As can be seen, 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 predictions is very small. Particularly, case 2 results for both MB and PSB for P215-M2110 (**Fig. 6.28(e)**) are exactly the same as case 1. Therefore, it can be concluded that the horizontal restraint along the edges of slab outstand has little effect on the vertical deflections of the edge beams.

a) P215-M1099

b) P368-M1099

c) P486-M1099

d) P215-M1356

e) P215-M2110

Fig. 6.28 Comparison of predicted and measured deflections of the edge beams – Series II

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 the simulations and the test results showed a ‘run-away’ behaviour due to significant reduction in strength and stiffness of steel and concrete, the predictions give smaller deflections than the tests. It is possibly due to the occurrences of concrete cracks above the edge beams. When these cracks had appeared, 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 through-out the fire durations, even though the strength and stiffness of the materials deteriorated with increasing temperature. Therefore, the predictions gave smaller deflections than the tests when the beam temperature reached 700°C. This is a drawback of the numerical model.

It can be observed that the results for the PSB are often worse than the MB. This may be caused by horizontal displacement of the supporting columns. Connected to the weak direction of the columns, the ends of PSB displaced inwards more than those of MB, leading to a greater additional deflection of the PSB compared to that of the MB. In the numerical model, meanwhile, the supporting columns were modelled by only imposing vertical restraint at the column locations.

6.4.4 Failure modes

The failure point could not be clearly captured in the FE model. Therefore, in order to compare model predictions with the test results, failure of composite slab-beam systems is assumed when one or a combination of the following criteria is met, which are based on BS 476 provisions ((BSI) 1987). It stipulates that failure of a bending member (a slab or a beam) occurs if (a) the deflection exceeds $L/20$, or (b) the 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. For the slabs, the limiting deflection ($L/20$) is calculated to be 113mm, while the second deflection limit ($L/30$) and the deflection rate are 75mm and 9.9mm/min, respectively. For the main beams, the corresponding values are 105mm, 70mm and 3.7mm/min, respectively; these values are 112mm, 75mm and 5.5mm/min for the protected secondary beams. When applying the two failure criteria to the measured deflections, 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 used.

Table 6.6 summarises the measured and predicted temperatures of the slabs at the ‘failure’ point. Temperatures of reinforcing mesh in the simulation at the ‘failure’ point are compared with the measured temperatures. As can be seen, a good correlation between the measured and predicted temperatures is obtained with the test/prediction ratio lying between 0.83 and 1.10 with a mean of 0.98.

Table 6.6 Measured and predicted temperatures of the slabs at failure

<i>Test</i>	<i>BS 476</i>	<i>Temperature at failure (°C)</i>		<i>Test/FEA</i>
	($L/20$)	<i>Test</i>	<i>FEA</i>	
<i>P215-M1099</i>	113	348	348	1.00
<i>P368-M1099</i>	113	316	381	0.83
<i>P486-M1099</i>	113	412	413	1.00
<i>P215-M1356</i>	113	351	359	0.98
<i>P215-M2110</i>	113	508	463	1.10
<i>M =</i>				0.98

Table 6.7 shows the comparisons between the measured and predicted deflections of the edge beams at the ‘failure’ point. The beam deflections from the simulations are compared to the test results at 113mm (L/20 deflection criterion). As can be seen, the ratio test/FEA lies between 0.86 and 1.33 with a mean of 1.11.

Table 6.7 Measured and predicted deflections of the edge beams at the slab deflection of 113mm

<i>Test</i>	<i>Member</i>	<i>Deflection at ‘failure’ (mm)</i>		<i>Test/FEA</i>
		<i>Test</i>	<i>FEA</i>	
<i>P215-M1099</i>	<i>MB</i>	45	47	0.96
	<i>PSB</i>	65	54	1.21
<i>P368-M1099</i>	<i>MB</i>	39	42	0.94
	<i>PSB</i>	68	54	1.25
<i>P486-M1099</i>	<i>MB</i>	40	47	0.86
	<i>PSB</i>	75	58	1.28
<i>P215-M1356</i>	<i>MB</i>	31	35	0.89
	<i>PSB</i>	69	64	1.07
<i>P215-M2110</i>	<i>MB</i>	25	20	1.27
	<i>PSB</i>	55	41	1.33
<i>M =</i>				1.11

To further validate the model, the observed and predicted final crack patterns on the slab top surface are compared and shown in **Fig. 6.29**. It is observed that the majority of cracks were in the vicinity of the supporting edge beams, around the support columns or over the top of the interior beams. This conforms to the experimental observations. Therefore, it can be concluded that the FE model is able to capture the crack patterns of the concrete slabs.

In the numerical model no attempt was used to predict the appearance of crack, therefore the cracks cannot be shown clearly. However, the Concrete damaged plasticity model allows observing the crack direction by plotting the maximum in-plane principal plastic strain (PE) at the integration points.

a) P215-M1099

b) P368-M1099

c) P486-M1099

d) P215-M1356

e) P215-M2110

Fig. 6.29 Comparison of observed and predicted final crack patterns – Series II

6.5 Discussions

To highlight the effect of the stiffness of the edge beams on the system behaviour, discussions are given separately for the stiffness of protected secondary edge beams (PSB), and for the stiffness of protected main beams (MB).

6.5.1 Distribution and development of membrane stresses against mesh temperature

Specimen P215-M1099 is chosen to study the distribution and development of membrane stresses against mesh temperature.

6.5.1.1 Distribution and development of membrane stresses in the slab and the beams

Figs. 6.30 and **6.31** plot the distribution of membrane stresses in the slab and in the main beam (MB), secondary edge beam (PSB) and interior beam (USB) at three stages: (i) after the loading phase (mesh temperature is 26°C), (ii) at a slab deflection of $1h_s$ (h_s is the slab thickness) at which the mesh temperature is 181°C , and (iii) at ‘failure’ where the slab deflection is 113mm and the mesh temperature is 348°C . SSAVG1 and SSAVG2 are the membrane stresses (Abaqus/CAE 2009) in the directions parallel to the main beam (SSAVG1) and the protected secondary beam (SSAVG2), respectively. In these figures, negative/positive values indicate compressive/tensile stresses towards/outwards the slab centre, respectively as shown in **Fig. 6.30(a)** and **Fig. 6.31(a)**.

(a) Membrane stress SSAVG1 in the slab after loading phase (Mesh temperature is 26°C)

(b) Membrane stress SSAVG1 in MB after loading phase (Mesh temperature is 26°C)

(c) Membrane stress SSAVG1 in the slab at a slab deflection of $1h_s$ (Mesh temperature is 181°C) (d) Membrane stress SSAVG1 in MB at a slab deflection of $1h_s$ (Mesh temperature is 181°C)

(e) Membrane stress SSAVG1 in the slab at 'failure' – a slab deflection of 113mm (Mesh temperature is 348°C)

(f) Membrane stress SSAVG1 in MB at 'failure' – a slab deflection of 113mm (Mesh temperature is 348°C)

Fig. 6.30 Distribution of membrane stresses SSAVG1 – P215-M1099

(a) Membrane stress SSAVG2 in the slab after loading phase (Mesh temperature is 26°C)

(b) Membrane stress SSAVG2 in PSB and USB after loading phase (Mesh temperature is 26°C)

(c) Membrane stress SSAVG2 in the slab at a slab deflection of $1h_s$ (Mesh temperature is 181°C)

(d) Membrane stress SSAVG2 in PSB and USB at a slab deflection of $1h_s$ (Mesh temperature is 181°C)

(e) Membrane stress SSAVG2 in the slab at 'failure' – a slab deflection of 113mm (Mesh temperature is 348°C)

(f) Membrane stress SSAVG2 in PSB and USB at at 'failure' – a slab deflection of 113mm (Mesh temperature is 348°C)

Fig. 6.31 Distribution of membrane stresses SSAVG2 – P215-M1099

Distribution of membrane stresses in the direction parallel to the main beam (SSAVG1)

At the end of the loading phase, in the direction parallel to the MB, compressive membrane stresses exist in almost the whole surface of the slab, except the vicinity of the PSB (**Fig. 6.30(a)**). This is because the slab resists the load through bending, and there is a small hogging moment above the PSB. The MB is in bending with the neutral axis (n.a.) within the top flange of the MB (**Fig. 6.30(b)**).

When the mesh temperature reaches 181°C , the compression ring is formed at a slab deflection of $1h_s$. At that temperature, there is an obvious tensile region at the slab centre with a width of 750mm (**Fig. 6.30(c)**). On the other hand, above the main beam, the slab acts as the T-flange for the main beam, and therefore part of the concrete slab is in compression. The width of the T-flange in compression is 825mm. The main beam is subjected to a significant bending moment caused by the load and temperature. Therefore, the n.a. of the MB shifts downwards and is within the beam web (**Fig. 6.30(d)**).

As the mesh temperature increases from 181°C to 348°C , the bending moment due to temperature in the MB increases. However, the strength and stiffness of both the concrete slab and the MB are reduced. As a result, a greater width of the T-flange is required to balance tensile stresses in the steel main beam. Therefore, the width of the T-flange slab increases from 825mm to 910mm, leading to a small decrease of the tensile region, from 750mm to 665mm (**Fig. 6.30(e)**). Due to the reduction of the strength, the tensile stresses in the MB reduce, and therefore the n.a of the MB shifts upwards and is within the top flange of the MB (**Fig. 6.30(f)**).

Distribution of membrane stresses in the direction parallel to the protected secondary beam (SSAVG2)

The distribution of membrane stresses in the direction parallel to the PSB (SSAVG2) is different as shown in **Fig. 6.31**. At the end of the loading phase, the compressive membrane stresses exist in almost the whole surface of the slab, except the vicinity of MB (**Fig. 6.31(a)**). This is because the slab resists the load through bending, and

there is a small hogging moment above the MB. Both the PSB and USB are resisting the load by bending. The whole sections of both beams are in tension and the n.a. of these beams falls into the slab (**Fig. 6.31(b)**).

When the mesh temperature reaches 181°C , the compression ring is formed at a slab deflection of $1h_s$. At that temperature, there is a tensile region at the slab centre. Parts of the slab above the PSB and USB are in compression, because they act as T-flanges for the composite secondary beams (PSB and USB) (**Fig. 6.31(c)**). Due to a significant bending moment caused by temperature, the n.a. of the PSB and the USB shifts downwards and is within the beam web (**Fig. 6.31(d)**).

As temperature increases, the effect of thermal bowing in the PSB and USB increases, resulting in a greater bending moment in the beams. On the other hand, the strength and stiffness of both the concrete slab and the beams are reduced. The PSB is in bending and its n.a. is in the beam top flange. The USB is in tension in the whole section (**Fig. 6.31(f)**). A greater part of T-flange concrete slab is required to balance tensile stresses in the PSB and USB. As a result, the compressive membrane stresses exist in the whole surface of the slab, and no clear tensile region can be observed at “failure” (**Fig. 6.31(e)**).

6.5.1.2 Development of membrane stresses across the slab mid-spans

Fig. 6.32 plots the development of membrane stresses at three stages: after the loading phase (26°C), at a slab deflection of $1h_s$ at which the mesh temperature is 181°C , and at ‘failure’ when the slab deflection is 113mm and the mesh temperature is 348°C . The membrane stresses in the directions parallel to the MB (SSAVG1) and the PSB (SSAVG2) are plotted across Section 2 and Section 1, respectively. In this figure, negative/positive values indicate compressive/tensile stresses towards/outwards the slab centre, respectively. Sections 1 and 2 are the sections across the slab mid-spans as shown in **Fig. 6.30(a)**.

Similar observations to **Section 6.5.1.1** can be drawn. At the end of the loading phase (mesh temperature is 26°C), compressive membrane stress exists along the mid-span sections (both Sections 1 and 2) because the slab is in bending.

Across Section 2 (**Fig. 6.32(a)**), when the compression ring is formed at a slab deflection of $1h_s$ at a mesh temperature of 181°C , there is a clear tensile zone at the slab centre with a width of 825mm. As the mesh temperature increases from 181°C to 348°C , a greater T-flange is required to balance the tensile stresses in the steel main beam. Therefore, the compressive region increases from 825mm to 910mm.

Across Section 1 (**Fig. 6.32(b)**), when the mesh temperature reaches 181°C , due to the bending moment in the PSB and USB caused by temperature and the load, parts of the slab above the PSB and USB are in compression. This is because they act as a T-flange for the composite secondary beams (PSB and USB). As the mesh temperature increases from 181°C to 348°C , the effect of thermal bowing in PSB and USB increases, resulting in a greater bending moment in the beams. Therefore, the magnitude of compressive membrane stress SSAVG2 increases, and no clear tensile region can be observed at “failure”.

a) Development of membrane stress SSAVG1 across Section 2

b) Development of membrane stress SSAVG2 across Section 1

Fig. 6.32 Development of membrane stresses across mid-span sections – P215-M1099

6.5.2 Effect of stiffness of protected secondary edge beams

6.5.2.1 Comparison of temperature distribution of the slabs

Fig. 6.33 Temperature distribution across slab thickness – Group 1

A comparison of temperature distribution of the specimens in *Group 1* is shown in **Fig. 6.33**. It can be observed that the air temperature in Group 1 developed consistently. 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. On the other hand, temperature at the slab bottom surface increased at the same rate up to 60min of heating. After that, as the temperature increased gradually concrete cracks developed resulting in significant heat loss. Therefore, towards the end of the tests, the temperature at the bottom surface and at the reinforcing mesh showed some discrepancies. Temperature at the top surface was very consistent with the maximum value of 150°C in P368-M1099.

6.5.2.2 Vertical deflection at the slab centre

The behaviour of the slabs in Group 1 against the mesh temperature is plotted in **Fig. 6.34**. It should be noted that the bending stiffness about the major axis of PSB (EI_{yPSB}) of P368-M1099 and of P486-M1099 had increased 1.71 and 2.26 times compared to that of control specimen P215-M1099.

Fig. 6.34 Comparison of deflection at the slab centre – Group 1

As can be seen, P215-M1099 failed at a deflection of 124mm when the mesh temperature had reached 348°C. The corresponding values were 118mm at 316°C, and 139mm at 412°C for P368-M1099 and P486-M1099, respectively. It can be observed that as the stiffness of PSB increased, the slab deflection decreased. In P368-M1099 test, this trend can also be observed up to a mesh temperature of 280°C. Beyond this temperature, P368-M1099 experienced greater deflections than P215-M1099 although the PSB of P368-M1099 had a greater stiffness. This was because after 71min of heating (from 280°C onwards) one significant crack appeared

above one of the main beams of P368-M1099 as shown in **Fig. 6.29(b)**. Consequently, composite action between the main beam and the concrete slab was weakened and the beneficial effect of increasing the stiffness of PSB vanished.

Table 6.8 Summary of experimental results of Group 1 – Series II

Specimen	h_s	Failure time	Failure temperature (°C)			p_{test}	$p_{y, \theta_{mesh}}$	w_{max}	w_{max} / h_s	$p_{test} / p_{y, \theta_{mesh}}$
			θ_{top}	θ_{mesh}	θ_{bottom}					
P215-M1099	57	72.2	108	348	602	15.6	8.4	124	2.17	1.86
P368-M1099	58	74.6	118	316	688	15.3	8.2	118	2.03	1.87
P486-M1099	55	98.2	127	412	697	15.5	8.2	139	2.53	1.89

With regard to the maximum deflection at failure, as can be seen in **Table 6.8**, P486-M1099 (the stiffness of PSB increased 2.26 times compared to that of P215-M0119) experienced larger deflections, i.e. $2.53h_s$ compared to $2.17h_s$ (h_s is the slab thickness), which was an increase of 17%. As a result, a greater enhancement factor was obtained, 1.89 for P486-M1099 compared to 1.86 for P215-M1099. This means that higher tensile membrane forces were mobilised in P486-M1099 than in P215-M1099. The enhancement factor is defined here as the ratio between the test load p_{test} and the yield line load at failure mesh temperature $p_{y, \theta_{mesh}}$.

However, an identical raise 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 had increased 1.71 times compared to that of P215-M1099. As explained above, this was due to weakening of the composite action between the main beam and the concrete slab after 71min of heating.

Therefore, it can be concluded that an increase of the stiffness of protected secondary edge beams had an initial positive effect on minimising the slab deflection. 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 was weakened by parallel cracks along the protected secondary beams, the floor

system would lose the benefit associated with a greater stiffness of secondary edge beams.

6.5.2.3 Mid-span deflection of the edge beams

Fig. 6.35 shows the mid-span deflection against temperature at the bottom flange of the protected edge beams. As expected, the deflection of PSB decreased when its stiffness increased, while the deflections of MB were almost similar in all three tests because the main beam size was the same.

Fig. 6.35 Comparison of mid-span deflection of the edge beams – Group 1

6.5.2.4 Tensile membrane region

Fig. 6.36 and **Fig. 6.37** plot the membrane forces from the numerical analyses across Sections 1 and 2 at the “failure” deflection of 113mm, in which negative and positive values indicate compressive and tensile forces, respectively. Sections 1 and 2 are the sections across the slab mid-span in both directions as shown in **Fig. 6.37**. The membrane forces across the midspan sections are the reaction forces taken from the nodes along the two axes of symmetry. It is noteworthy that due to symmetry, only quarter of the slab was modelled. The actual failure point determined from the tests cannot be used for comparison here, because each specimen failed at a different time with a different deflection. The effect of the stiffness of PSB should be compared numerically at a similar deflection of the slabs. Further discussions on the distribution and development of membrane forces are given in **Section 6.5.1**.

No clear tensile region can be observed in **Fig. 6.36**. Across Section 1, there exists only compressive forces; across Section 2, there are clearly tensile region and a compression ring. The width of tensile region is about 750mm for all three specimens. The compression ring consists of part of the slab above the edge beams and part of the slab outstands due to T-flange effect of the edge beams as can be seen in **Fig. 6.37**. However, there is no obvious relationship between the stiffness of the secondary edge beams and the dimension of the tensile region.

Fig. 6.36 Traction / Compression force across the mid-spans of the slabs at a deflection of 113mm – Group 1

Fig. 6.37 Typical distribution of membrane forces – Group 1

6.5.3 Effect of stiffness of protected main beams

6.5.3.1 Comparison of temperature distribution of the slabs

Fig. 6.38 shows the temperature distribution of the specimens in Group 2. The air temperature of P215-M1099 and P215-M1356 were very close with the maximum value of 973°C in P215-M1099 compared to 963°C in P215-M1356. However, in P215-M2110 test, unfortunately the air temperature increased at a very slow rate after 28min. Temperature reached a maximum value of only 820°C after 198min at which the slab failed. An investigation after P215-M2110 test showed that a heating element was damaged by concrete spalling. Consequently, in P215-M2110 temperatures at the slab bottom surface and at the reinforcing mesh also increased at a very slow rate. Even though the test results could be used to simulate the FE model, numerical investigation with the same parameters of P215-M2110 could be conducted.

Fig. 6.38 Temperature distributions across slab thickness – Group 2

6.5.3.2 Deflection at the slab centre

The central deflection of the slabs in Group 2 against mesh temperature is plotted in **Fig. 6.39**. It should be noted that the bending stiffness about the major axis of the main beam (EI_{yMB}) of P215-M1356 and of P215-M2110 had increased 23% and 92% compared to EI_{yMB} of control specimen P215-M1099.

Fig. 6.39 Comparison of central deflection of the slabs – Group 2

The experimental result of P215-M2110 cannot be used to compare directly because its heating rate did not conform to the other two tests. Based on the validated numerical model, using the material and geometry properties of P215-M2110 but the fire curve of P215-M1356, the predicted test result of P215-M2110 can be obtained as shown in **Fig. 6.39**. This helps to eliminate the discrepancy due to the low heating rate in P215-M2110 test.

It can be observed that as the stiffness of MB increased, the slab deflection decreased. When the mesh temperature was smaller than 100°C, the difference was small. This was because the temperature was still small and the behaviour was mainly due to the load. As temperature increased, the effect of EI_{yMB} became more obvious. At a mesh temperature of 200°C, with an increment of 23% of EI_{yMB} , the deflection reduced 24% (50mm of P215-M1356 compared to 66mm of P215-M1099). Correspondingly, with an increment of 92% in EI_{yMB} , the deflection reduced 44% (36mm of P215-M2110 compared to 66mm of P215-M1099). Obviously, increasing the stiffness of the main beam reduced the slab deflection.

However, when the mesh temperature reached 300°C, the results of P215-M1356 converged to those of P215-M1099, and the failure point was very close. This was due to severe cracks occurring above the main beam in P215-M1356 when its mesh temperature reached 300°C (**Fig. 6.29(d)**).

Table 6.9 summarises the experimental results of Group 2 tests. P215-M1099 failed at a deflection of 124mm when the mesh temperature had reached 348°C. P215-M1356 failed at a deflection of 121mm at a mesh temperature of 351°C. P215-M2110 failed at a deflection of 143mm at a mesh temperature of 508°C. No obvious relationship between the maximum deflection of the slabs and the values of EI_{yMB} was observed. This was because as cracks in the vicinity of the main edge beams developed, the composite action between the beams and the concrete slab decreased. As a result, the beneficial effect of increasing the stiffness of MB on the slab deflection vanished.

Table 6.9 Summary of experimental results of Group 2 – Series II

Specimen	h_s mm	Failure time min	Failure temperature (°C)			P_{test} kN/m ²	$P_{y, \theta_{mesh}}$ kN/m ²	w_{max} mm	w_{max} / h_s	$P_{test} / P_{y, \theta_{mesh}}$
			θ_{top}	θ_{mesh}	θ_{bottom}					
P215-M1099	57	72.2	108	348	602	15.6	8.4	124	2.17	1.86
P215-M1356	58	76.3	101	351	734	15.4	8.7	121	2.09	1.77
P215-M2110	59	198.0	228	508	715	15.6	8.3	143	2.42	1.88

Therefore, it can be concluded that the stiffness of the main beams was unlikely to enhance TMA in the slab. The failure mode at tensile membrane stage of a floor assembly was mainly controlled by the compression ring forming at the peripheral and the tensile region at the centre of the slab.

6.5.3.3 Mid-span deflection of the edge beams

Fig. 6.40 shows a comparison of mid-span deflection of the protected edge beams against their temperature at the bottom flange. A similar trend can be observed. Initially, the edge beams gradually deflected up to 600°C. Thereafter, from 600°C onwards, the beams deflected at a significantly greater rate because they gradually lost the fire-proofing.

Fig. 6.40 Comparison of mid-span deflection of the edge beams – Group 2

As expected, MB of P215-M1099 (noted as ‘P215-M1099 MB’ in the caption) had the greatest deflection since its bending stiffness was the smallest. On the other hand, the deflection of P215-M2110 MB was only slightly smaller than that of P215-M1356 even though the stiffness of P215-M2110 MB was greater than 55%. This may be due to the slower heating rate of P215-M1356 compared to that of P215-M1356.

6.5.3.4 Tensile membrane action

The membrane forces from the numerical analyses across Sections 1 and 2 at the ‘failure’ deflection of 113mm are plotted in **Fig. 6.41**. In this figure, negative and positive values indicate compressive and tensile forces, respectively. It should be noted that the deflection of 113mm was assumed to be the failure deflection in the numerical analyses. Section 1 and Section 2 are the sections across the slab mid-span as shown in **Fig. 6.25**.

a) Section 1

b) Section 2

Fig. 6.41 Traction / Compression force across the mid-spans of the slabs at a deflection of 113mm – Group 1

No clear tensile region can be found. Across Section 1, there exists only compressive forces; across Section 2, there is a clear tensile region and a compression ring. The width of tensile region is 725mm, 600mm, and 355mm for P215-M1099, P215-M1356, and P215-M2110, respectively. As the stiffness of the main beam increases, the tensile region becomes smaller. This is possibly because as the stiffness of the main beam increases, part of the concrete slab, which plays as a T-flange for the composite main beam, is extended to balance with greater tensile force produced in that main beam. As a result, the width of tensile region at the slab centre is reduced.

6.6 Conclusions

This chapter presents the experimental results and observations on five composite beam-slab systems tested in fire with the aim to study the effect of bending stiffness of the protected main and secondary edge beams on the behaviour of the slabs. Based on the test results and numerical models, the following conclusions can be drawn:

- (1) As the stiffness of the protected edge beams increased (main or secondary edge beams), the slab deflection initially decreased. However, the

experimental tests showed that composite action between the edge beams and the concrete slab plays a key role in mobilising this beneficial effect. Once the composite action had 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 approximately 0.9 to 1.0 of the slab thickness, irrespective of the bending stiffness of the edge beams. The imminence 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 displacements along the slab edges; and (c) horizontal and vertical displacements of protected steel columns. In all the tests, these indicators were very consistent in terms of occurrence time.
- (3) The tests revealed different slab-beam failure modes of the floor assemblies. In all specimens, severe cracks appeared in the vicinity of the protected edge beams, resulting in fracture of reinforcement at these regions.
- (4) The composite floor systems with unprotected interior beams can definitely mobilise tensile membrane action in fire. However, no clear tensile region at the slab centre can be found. Across the section which is perpendicular to the secondary beams, there are only compressive forces in the slab. This is because part of the concrete slab acts as a T-flange for the composite secondary beam, which balances the tensile forces in the steel beams. On the other hand, as the stiffness of the main beam increases, the width of the tensile region is reduced, because a larger T-flange is mobilised.
- (5) The numerical model was shown to be able to predict accurately the thermal/structural behaviour of composite slab-beam systems subjected to fire conditions. However, the model assumed perfect bonding between the steel beam and the concrete slab. Therefore, it could not simulate accurately deterioration of full composite action between the steel downstand beams and the slab as temperature increases and as cracking took place. This is a shortcoming of the proposed numerical model.

CHAPTER 7 SEMI-ANALYTICAL METHOD TO PREDICT THE ULTIMATE LOAD OF UNRESTRAINED SLAB-BEAM SYSTEMS IN FIRE

7.1 Introduction

At ambient temperature, the applied load on the slab in a composite floor system is distributed from the slab to the secondary beams in one-way action. The load path involved in resisting permanent and variable loads under ambient condition is: slab → secondary beams → main beams → columns.

Under severe fire conditions, if the secondary interior beams are unprotected, due to degradation of strength in fire they lose most of their strength. As a result, the beams form plastic hinges and the load path at ambient temperature cannot be maintained. The load-carrying mechanism changes to a two-way bending system. The region involving the slab and the unprotected interior beams is known as *a slab panel*. Under severe fire conditions, the load applied to a slab panel is distributed in a load path as follows: slab → supporting edge beams → columns.

The slab panel develops its load-bearing capacity in the deformed state through a combination of yield-line mechanism and tensile membrane action. A number of analytical studies have attempted to predict the enhanced load-bearing capacity of slab due to TMA as discussed in **Section 2.5.1**. Among these approaches, a well-known and user-friendly approach, namely, the Bailey-BRE method (Bailey and Moore 2000a), has been adopted in the SCI Publication P288 (Newman *et al.* 2006) and applied in the UK practice. However, the method does not provide any information on the deflection of protected edge beams, apart from the assumption that their deflections are negligible throughout a fire.

In order to propose a semi-analytical approach which can consider the behaviour of edge beams, discussions on the Bailey-BRE method are given in **Section 7.2**. The

proposed approach is presented in **Section 7.3**. **Section 7.4** presents the calibration of the model against test Series I and II. Finally, conclusions are given in **Section 7.5**.

7.2 Discussions on the Bailey-BRE Method

The Bailey-BRE method begins by dividing a composite floor system into several horizontally-unrestrained, vertically supported slab panels – *floor design zones* (**Fig. 7.1**). Each of these floor design zones consists of simply-supported unprotected interior beams. As temperature increases, the formations of plastic hinges in the interior beams re-distribute the loads to the two-way bending slab which undergoes large vertical deflections. Based on rigid-plastic theory with large change of geometry, the additional slab capacity provided by TMA is calculated as *an enhancement to the small-deflection conventional yield-line capacity* of the concrete slab panel.

Fig. 7.1 Typical slab panels

The derivation of the Bailey-BRE method (Bailey and Moore 2000a) is given in **Appendix C**. By assuming that under tensile membrane stage, the dominant load-carrying capacity of the system is due to the composite slab, the following simplifying assumptions have been used in the Bailey-BRE method:

1. The slab is assumed to be simply supported irrespective of geometric configurations, and the edges are unrestrained from planar movement. This

implies that TMA is formed in the central region of the slab and a compressive ring is formed around the perimeter of the slab (**Fig. 2.7**). This is the self-equilibrating mechanism of TMA mobilised in *an unrestrained isolated slab panel*.

2. The load carried by the flexural behaviour of the grillage of composite beams, within the fire compartment, is based on the lower-bound mechanism for the beam with the highest load ratio (*i.e.* the beam which will ‘fail’ first in the fire). The beams are assumed simply-supported and will support a loaded area calculated assuming the slab is simply supported (*i.e.* typical assumptions taken for normal design).
3. The load supported by the flexural behaviour of the composite slab is calculated based on the lower-bound yield-line mechanism, assuming that the unprotected interior beams have zero resistance.
4. The enhancement due to membrane action in the composite slab is based on the lower bound yield-line mechanism of the slab.
5. The load carrying capacity of *the composite interior beams and the slab* (enhanced due to membrane action) are added together.

The first question arises as to whether the heated panels can be considered unrestrained or restrained against horizontal movement. As shown in **Chapters 5 and 6**, in all the tests the slab reinforcement had fractured above the edge beams of the slab. This suggests that the heated slab panels should be considered as unrestrained against horizontal movement at tensile membrane stage. Therefore, this assumption is valid.

The second question also is based on the first assumption that vertical supports along the slab panel boundaries remain rigid at all times during a fire. To provide the necessary vertical support the edge beams of a slab panel must be protected to a required fire resistance. This is essential for the slab to bend in two-way curvature.

However, in reality the edge beams may deform considerably even though they are fire-protected, and thus this assumption may be nullified. On the other hand, even if

the edge beams deflect but have not formed plastic hinges yet, the composite beam-slab collapse mechanisms do not occur. As a result, TMA can be still mobilised at a certain level. Therefore, in the proposed approach the author tries to estimate the load-bearing capacity of the slab which is enhanced by TMA but reduced by the deflecting edge beams.

The last question is based on the fifth assumption. When calculating the contribution of unprotected interior beams to the total load-bearing capacity of the slab, Bailey treated these beams as composite beams. Under tensile membrane stage, since the load path is changed (slab $\rightarrow$ supporting edge beams $\rightarrow$ columns), if the interior beams are treated as composite beams, it may overestimate the total load capacity by taking into account twice the load supported by parts of concrete slab over the interior beams. This assumption is therefore verified against the author's test results in **Section 7.5**.

In the simple design method, a deflection limit has to be assumed, and then the enhancement above the yield-line load bearing capacity of the slab due to TMA can be calculated based on that deflection. Discussions and validation of the deflection limit proposed by Bailey and Moore (2000a) are given in **Section 8.3**.

7.3 Proposed Semi-Analytical Approach

7.3.1 Assumptions

The following assumptions are adopted in the proposed model, which are based on the author's understanding of floor panel behaviour in fire and the eight fire tests conducted.

- (1) The slab panel is assumed to be rectangular in plan.
- (2) The slab panel is assumed to be unrestrained against horizontal movement, and supported along four edges by the protected beams.

- (3) The load supported by the flexural behaviour of the composite slab is calculated based on the lower-bound yield-line mechanism, assuming that the interior beams have zero resistance.
- (4) The protected edge beams are assumed to be simply supported without any lateral restraint.
- (5) The load distribution on the edge beams at collapse follows the shape of the segments of the yield line mechanism, *i.e.* triangular loading distribution for the short-span beams and trapezoidal loading distribution for the long-span beams.
- (6) Torsional rigidity of the edge beams is assumed to be negligible.
- (7) The load carrying capacity of *the steel interior beams and the slab* (enhanced due to membrane action) are added together.

As discussed in **Section 7.2**, based on the author's test results, the 2nd assumption is accurate and conservative. Therefore, the slab panel is considered unrestrained against horizontal movement in the proposed model.

When calculating the load capacity of the slab, the proposed model uses the 2nd assumption, in which the lower-bound yield-line mechanism in the slab is adopted. This is a shortcoming of the proposed model since it cannot take into account the interaction between the slab and the unprotected interior beams. The contribution of the unprotected interior beams on the load capacity of the composite beam-slab system in fire is considered by using the 7th assumption.

The 6th assumption is a reasonable assumption since the numerical analyses in **Chapter 3** have shown that the effect of torsional rigidity of protected edge beams on the slab behaviour is not significant.

When calculating the additional load supported by unprotected interior beams in fire, the interior beams are assumed to be bare steel beams (the 7th assumption). To validate this assumption, comparisons between the proposed model and the test results are conducted with two cases as presented in **Section 7.4**. Case 1 treated the

interior beams as composite beams, and Case 2 treated the interior beams as bare steel beams.

7.3.2 Failure modes

A composite floor system may fail due to failure of the slab panel alone or due to the composite beam-slab collapse mechanisms which involve failure of both the panel and the beams. These two failure mechanisms, *i.e.* *single slab panel* (**Fig. 7.2**) and *beam-slab collapse mechanisms* (**Fig. 7.3**), are the two extreme modes of mechanism. The failure modes observed in the author's tests were somewhat in between these two modes.

In terms of the failure of the slab panel alone, Bailey and Toh (2007a) proposed two failure modes as shown in **Fig. 7.2**, which are based on their fire tests conducted on isolated slabs. The first failure mode is due to fracture of the mesh reinforcement across the short span at the centre of the slab. A second failure mode occurs due to crushing of the concrete in the corners of the slab where high compressive in-plane forces develop.

Fig. 7.2 Assumed failure modes for isolated slab panels (Bailey and Toh 2007a)

In the author's experiment, the second failure mode (**Fig. 7.2(b)**) was observed in the S3-FR test (**Section 5.3.7**). In the other seven tests, although there were cracks at the centre of the slab, fracture of the mesh did not occur. This was because the tests were

stopped when full-depth cracks had occurred above the protected edge beams. If the tests were to be continued, fracture of the mesh at the slab centre might have occurred. Therefore, the author uses these two failure modes of the slab panel (**Fig. 7.2**) in estimating the enhancement factor of the slab due to TMA.

The composite beam-slab collapse mechanisms may appear if plastic hinges form in the perimeter beams and the yield lines occur across the centre of the slab as shown in **Fig. 7.3**. These collapse mechanisms were proposed by Abu *et al.* (2010) and have been included in SCI Publication P390.

Fig. 7.3 Composite collapse mechanisms (Abu *et al.* 2010)

The most important condition for the mobilisation of TMA is that the slab must bend in synclastic curvature. Once the 'folding' mechanism has formed, TMA will be vanished, leading to failure of the system. Therefore, the edge beams must be checked to ensure a minimum required moment resistance in order that the composite collapse mechanisms do not occur. Verification of the resistance of the edge beams is discussed in **Section 7.3.5**.

7.3.3 Overall enhancement factor

Based on the yield line pattern, the slab is divided into four rigid bodies as shown in **Fig. 7.4**. The combined enhancement factor for each element proposed by Bailey and Moore (2000) is as follows:

$$\begin{aligned} e_1 &= e_{1m} + e_{1b} \\ e_2 &= e_{2m} + e_{2b} \end{aligned} \quad (7.1)$$

where e_{1m} and e_{2m} are the contribution of membrane forces to load bearing capacity of elements 1 and 2, respectively; e_{1b} and e_{2b} are the factors which take into account the effect of membrane forces on the bending resistance due to the presence of axial force of elements 1 and 2, respectively.

Fig. 7.4 Yield-line pattern

The values e_1, e_2 are not the same. Hayes (1968a) suggested that these differences can be explained by the effect of the vertical or in-plane shear and the overall enhancement is thus given by:

$$e = e_1 - \frac{e_1 - e_2}{1 + 2\mu a^2} \quad (7.2)$$

where a is the aspect ratio of the slab (L/l); μ is the ratio of the yield moment capacity of the slab in orthogonal directions.

In the Bailey-BRE method, the values e_1 and e_2 are calculated based on the equilibrium of elements 1 and 2 with *the vertical rigid supports*. As a result, the method has to assume that the supports along the edges of the slab panel remain rigid. Therefore, it cannot consider vertical deflections of the edge beams.

The deformed shape of a slab-beam floor system is shown in **Fig. 7.5**. In order to take into account the behaviour of the edge beams, the author proposes a revised form of Eq. (7.1), as follows:

$$\begin{aligned} e_1^* &= e_{1m}^* + e_{1b} \\ e_2^* &= e_{2m}^* + e_{2b} \end{aligned} \quad (7.3)$$

and the overall enhancement is given by:

$$e^* = e_1^* - \frac{e_1^* - e_2^*}{1 + 2\mu a^2} \quad (7.4)$$

Fig. 7.5 Deformed shape of a slab-beam floor system

The predicted *load bearing capacity of the slab enhanced by TMA* is calculated by Eq. (7.5):

$$q_{s,\theta} = e^* \times p_{y,\theta} \quad (7.5)$$

where $p_{y,\theta}$ is the yield-load of the slab at temperature θ .

In the proposed approach, the values e_1^* and e_2^* are calculated based on the equilibrium of elements 1 and 2 with *the deformed supporting edges*. The deformations are determined based on the deflection shape of the edge beams. **Section 7.3.4** presents the derivations in detail.

7.3.4 Derivation

7.3.4.1 In-plane stress distribution

The inplane stress distribution is assumed to be similar to the Bailey-BRE method as shown in **Fig. 7.6**, which is assumed that the in-plane forces comprise compressive membrane forces around the perimeter of the slab and tensile membrane forces in the central area of the slab. The magnitude of the forces is defined by the constants k and b which depend on the dimensions of the slab. The derivations of the inplane stress distribution are conducted by Hayes (1968a) and Bailey and Moore (2000a) (**Appendix C**). Only the results are given below.

Fig. 7.6 In-plane stress distribution for membrane action

a) Parameters k and b

The parameters k and b are determined from equilibrating of the in-plane stress distribution.

$$k = \frac{4na^2(1-2n)}{4n^2a^2+1} + 1 \quad (7.6)$$

The intersection point of the yield lines is defined by the parameter n calculated using the yield-line theory:

$$n = \frac{1}{2\sqrt{\mu a^2}} \left[\sqrt{3\mu a^2 + 1} - 1 \right] \leq 0.5$$

n is limited to the maximum value of 0.5 resulting in a valid yield line pattern; a is the aspect ratio of the slab (L/l); μ is the ratio of the yield moment capacity of the slab in orthogonal directions (should always be less than or equal to 1.0).

The constant b is chosen as the minimum value in the following two expressions, which are associated with two failure modes of the slab as shown in **Fig. 7.2(a)** and **(b)**:

- *1st failure mode*: fracture of mesh reinforcement across the short span of the slab:

$$b = \frac{1.1l^2}{8K(A+B+C-D)} \quad (7.7)$$

- *2nd failure mode*: compressive failure of concrete

$$b = \frac{1}{kKT_0} \left[0.85f_{ck} 0.45 \left(\frac{d_1+d_2}{2} \right) - T_0 \left(\frac{K+1}{2} \right) \right] \quad (7.8)$$

where d_1 and d_2 are the effective depths in both orthogonal directions; the parameters A, B, C, D are the parameters calculated by the followings:

$$A = \frac{1}{2} \left(\frac{1}{1+k} \right) \left[\frac{l^2}{8n} - \frac{(L/2-nL)}{nL} \left((nL)^2 + \frac{l^2}{4} \right) - \frac{1}{3} \left(\frac{1}{1+k} \right) \left((nL)^2 + \frac{l^2}{4} \right) \right]$$

$$B = \frac{1}{2} \left(\frac{k^2}{1+k} \right) \left[\frac{nL^2}{2} - \frac{k}{3(1+k)} \left((nL)^2 + \frac{l^2}{4} \right) \right]$$

$$C = \frac{l^2}{16n}(k-1)$$

$$D = \left(\frac{L}{2} - nL\right)\left(\frac{L}{4} - \frac{nL}{2}\right)$$

T_0 and KT_0 are the resistance of the reinforcing mesh per unit width in long and short spans, respectively. The bending moments M_0 and μM_0 per unit width of the slab in long and short spans respectively can be calculated by:

$$\mu M_0 = KT_0 d_1 \left(\frac{3 + (g_0)_1}{4} \right) \quad (7.9)$$

$$M_0 = T_0 d_2 \left(\frac{3 + (g_0)_2}{4} \right) \quad (7.10)$$

where $(g_0)_1$ and $(g_0)_2$ are parameters which define the flexural stress block in short and long spans, respectively;

The yield-line resistance of the slab panel at a temperature θ can be calculated by Eq. (7.11).

$$p_y = \frac{24\mu M_0}{l^2} \left[\sqrt{3 + \frac{1}{(\sqrt{\mu a})^2} - \frac{1}{\sqrt{\mu a}}} \right]^{-2} \quad (7.11)$$

b) Membrane force along the yield line for Element 1

For the yield line BC (**Fig. 7.6**), the membrane force is constant and equals to:

$$N_x|_{BC} = -bKT_0 \quad (7.12)$$

Referring to **Fig. 7.7**, the membrane force across the yield line AB, at a distance of x from the centre point of the slab O is given by:

$$N_x|_{AB} = -bKT_0 + \frac{x - (L/2 - nL)}{nL} (k+1)bKT_0 \quad (7.13)$$

Fig. 7.7 Force applied to element 1, yield line AB

c) *Membrane force along the yield line for Element 2*

Referring to **Fig. 7.8**, the force at a distance y from B can be expressed as:

$$N_y = -bKT_0 + \frac{y}{l/2}(k+1)bKT_0 \quad (7.14)$$

Fig. 7.8 Force applied to element 2

7.3.4.2 Out-of-plane stress distribution

Based on the 4th assumption as proposed by Park and Gamble (1980), beam 1 (long-span beam) is subjected to a trapezoidal load with the maximum value $q_1 = ql/2$. Beam 2 (short-span beam) is subjected to a triangular load with the maximum value $q_2 = q(nL)$. The deformed shapes of beam 1 and beam 2 are $w_{zx}(x)$ and $w_{zy}(y)$, respectively. The deflection of the slab is w_m .

It is reasonable to assume that the shear forces along the yield lines are negligible. This is because Park and Gamble (1980) showed that at the junction of three yield lines (points B and C in **Fig. 7.6**) and at the intersection of a yield line and a free

edge (points A and D in **Fig. 7.6**), the nodal forces should be zero, provided that the mesh properties are identical in two orthogonal directions. Therefore, the out-of-plane stress distribution is as shown in **Fig. 7.9** and the derivations conducted by the author are given as follows.

Element 1 Element 2 Beam 1 Deformed shape of Beam 1 Beam 2 Deformed shape of Beam 2

Fig. 7.9 Out-of-plane stress distribution for membrane action

a) Deformed shape of Beam 1

The deflection of Beam 1 at elevated temperatures consists of two components: one from the trapezoidal load transferred from the slab and another from thermal gradient over the composite section of the edge beam. The first can be found based on mechanics while the latter can be calculated as follows. It should be noted that the beam is assumed to be simply supported without any lateral restraint.

Deflection due to thermal gradient over the beam depth: $w_{x\theta}(x)$

The thermal gradient over the total depth of Beam 1 is:

$$T_{,zb1} = \frac{T_{2b1} - T_{1b1}}{h_{b1}}$$

where T_{2b1} and T_{1b1} are the respective temperatures at the bottom and the top surfaces; h_{b1} is the total depth of beam 1 which is equal to $h_{b1} = h_{sb1} + h_s$, where h_{sb1} is depth of the steel beam 1, h_s is the slab depth.

Fig. 7.10 Simply supported beam subjected to a uniform thermal gradient

A uniform curvature $\phi = \alpha T_{,zb1}$ (α is thermal expansion) is induced along the length as a result of thermal gradient. For a simply supported beam without any lateral restraint subjected to a uniform curvature ϕ one can write:

$$\begin{aligned} \frac{d^2 w_{zx\theta}}{dx^2} &= \phi \rightarrow \frac{d^2 w_{zx\theta}}{dx^2} = \alpha T_{,zb1} \\ \rightarrow w_{zx\theta} &= \alpha T_{,zb1} \frac{x^2}{2} + C_1 x + C_2 \end{aligned} \quad (7.15)$$

With the boundary conditions: $w_{zx\theta} = 0$ at $x = \pm L/2$, one obtains:

$$w_{zx\theta}(x) = \alpha T_{,zb1} \left(\frac{L^2}{8} - \frac{x^2}{2} \right) \quad (7.16)$$

Deflection due to the applied load and the thermal gradient:

Referring to **Fig. 7.11**, it is easy to derive the deflection for Beam 1 due to the trapezoidal load. Using the superposition principle, the total deflection for beam 1 for each segment is given in Eqs. (7.17) and (7.18). The first two terms are the deflection due to the load, while the other terms are due to thermal gradient over the beam depth.

Fig. 7.11 Load transferred to Beam 1

Segment OB: $0 \leq x \leq L/2 - nL$

$$w_{zx}^{(1)} = w_{zx \max} - \frac{1}{EI_1} \left[M_{xO} \times \frac{x^2}{2} - q_1 \frac{x^4}{24} \right] + \alpha T_{,zbl} \left(\frac{L^2}{8} - \frac{x^2}{2} \right) \quad (7.17)$$

Segment BA: $L/2 - nL \leq x \leq L/2$:

$$w_{zx}^{(2)} = w_{zx \max} - \frac{1}{EI_1} \left[M_{xO} \times \frac{x^2}{2} - q_1 \frac{x^4}{24} \right] - \frac{1}{EI_1} \frac{q_1}{nL} \frac{(x - (L/2 - nL))^5}{120} + \alpha T_{,zbl} \left(\frac{L^2}{8} - \frac{x^2}{2} \right) \quad (7.18)$$

The maximum displacement $w_{zx \max}$ due to the load can be found from the boundary condition: $w_{zx}^{[2]} = 0$ at $x = L/2$

$$w_{zx \max} = \frac{q_1}{EI_1} \left[\frac{1}{24} \left[8(nL)^2 + 12(nL)(L - 2nL) + 3(L - 2nL)^2 \right] \times \frac{1}{8} L^2 - \frac{1}{24} \left(\frac{L}{2} \right)^4 + \frac{(nL)^4}{120} \right] \rightarrow w_{zx \max} = \frac{q_1}{EI_1} \alpha_1 \quad (7.19)$$

The maximum bending moment of the beam due to the load M_{xO} is:

$$M_{xO} = \frac{q_1}{24} \left[8(nL)^2 + 12(nL)(L - 2nL) + 3(L - 2nL)^2 \right] = \beta_1 q_1 \quad (7.20)$$

Substituting Eqs. (7.19) and (7.20) to (7.17) and (7.18), one obtains the deflection equation of Beam 1.

b) Deformed shape of Beam 2

Similarly, the deflection equation of Beam 2 due to thermal gradient over the total depth of Beam 2 is given by:

$$T_{,zb2} = \frac{T_{2b2} - T_{1b2}}{h_{b2}}$$

$$w_{zy\theta}(y) = \alpha T_{,zb2} \left(\frac{l^2}{8} - \frac{y^2}{2} \right) \quad (7.21)$$

Fig. 7.12 Load transferred to Beam 2

The deflection equation for Beam 2 due to both the load and the elevated temperatures is given in Eq. (7.22) with $0 \leq y \leq l/2$.

$$w_{zy}(y) = w_{zy\max} - \frac{1}{EI_2} \left[M_{yB} \times \frac{y^2}{2} - q_2 \frac{y^4}{24} + \frac{2}{l} q_2 \times \frac{y^5}{120} \right] + \alpha T_{,zb2} \left(\frac{l^2}{8} - \frac{y^2}{2} \right) \quad (7.22)$$

The maximum displacement $w_{zy\max}$ due to the load can be found by the boundary condition: $w_{zy} = 0$ at $y = l/2$.

$$w_{zy\max} = \frac{1}{120} \frac{q_2 l^4}{EI_2} \quad (7.23)$$

The maximum bending moment of the beam M_{yB} is given by:

$$M_{yB} = \frac{1}{12} q_2 l^2 \quad (7.24)$$

Substituting Eqs. (7.23) and (7.24) to (7.22), one obtains the deflection equation of Beam 2.

7.3.4.3 Contribution of membrane forces to load bearing capacity (e_{1m}^*, e_{2m}^*)

a) Element 1 (e_{1m}^*)

Taking the moments about the support of Element 1 (**Fig. 7.9**):

$$\int_{BC} N_x (w_m - w_{zx}^{(1)}) dx + 2 \int_{AB} N_x \left(\frac{L/2 - x}{nL} w_m - w_{zx}^{(2)} \right) dx - \text{Moment due to } \sum q = 0 \quad (7.25)$$

Substituting Eqs. (7.12) and (7.13) to (7.25):

$$\begin{aligned}
 & 2 \int_0^{L/2-nL} bKT_0 (w_m - w_{zx}^{(1)}) dx \\
 & + 2 \int_{L/2-nL}^{L/2} \left[bKT_0 - \frac{x - (L/2 - nL)}{nL} (k+1) bKT_0 \right] \times \left[\frac{L/2 - x}{nL} w_m - w_{zx}^{(2)} \right] dx \\
 & - q(L - 2nL) \frac{l}{2} \frac{l}{4} - q \times nL \times \frac{l}{2} \times \frac{l}{6} = 0
 \end{aligned} \quad (7.26)$$

It should be noted that $q_l = ql/2$. After integrating and regrouping (**Appendix D**), Eq. (7.26) becomes Eq. (7.27):

$$\begin{aligned}
 & 2bKT_0 \times \left[w_m \left(\frac{L}{2} - nL \right) - \delta_1 + (\bar{C} + \bar{E}) - \beta_3 \times \alpha T_{,zbl} \times \left[\frac{1}{8} L^2 (nL) - \frac{1}{6} \left(\left(\frac{L}{2} \right)^3 - \left(\frac{L}{2} - nL \right)^3 \right) \right] \right] \\
 & + \frac{k+1}{nL} \alpha T_{,zbl} \left(\frac{1}{16} L^2 (nL)^2 - \frac{1}{8} (nL)^4 \right) \\
 & = ql \left[\left(\frac{1}{8} - \frac{n}{6} \right) Ll + bKT_0 \frac{1}{EI_1} (\gamma_1 - \alpha_3) - bKT_0 \times \beta_3 \frac{1}{EI_1} \left(-\alpha_2 + \frac{(nL)^5}{720} \right) \right]
 \end{aligned} \quad (7.27)$$

where

$$\begin{aligned}
 \alpha_1 &= \frac{1}{24} \left[8(nL)^2 + 12(nL)(L - 2nL) + 3(L - 2nL)^2 \right] \times \frac{1}{8} L^2 - \frac{1}{24} \left(\frac{L}{2} \right)^4 + \frac{(nL)^4}{120} \\
 \beta_1 &= \frac{1}{24} \left[8(nL)^2 + 12(nL)(L - 2nL) + 3(L - 2nL)^2 \right] \\
 \gamma_1 &= \alpha_1 \left(\frac{L}{2} - nL \right) - \frac{\beta_1}{6} \left(\frac{L}{2} - nL \right)^3 + \frac{1}{120} \left(\frac{L}{2} - nL \right)^5 \\
 \delta_1 &= \alpha T_{,zbl} \left[\frac{1}{8} L^2 \left(\frac{L}{2} - nL \right) - \frac{1}{6} \left(\frac{L}{2} - nL \right)^3 \right] \\
 \alpha_2 &= \left(\alpha_1 \frac{L}{2} - \frac{\beta_1}{6} \frac{L^3}{8} + \frac{1}{120} \frac{L^5}{32} \right) - \gamma_1 \\
 \bar{C} &= w_m L \left[-\frac{1}{8n} + \frac{1}{2} + \frac{1}{2n} \left(\frac{1}{2} - n \right)^2 \right]
 \end{aligned}$$

$$\begin{aligned}\bar{E} &= -\frac{(k+1)}{(nL)^2} w_m \times L^3 \times \beta_2 \\ \beta_2 &= \frac{1}{48} - \frac{1}{8} \left(\frac{1}{2} - n \right) + \frac{1}{4} \left(\frac{1}{2} - n \right)^2 - \frac{1}{6} \left(\frac{1}{2} - n \right)^3 \\ \alpha_3 &= \frac{k+1}{nL} \left[\gamma_2 - \frac{1}{120} \left(\frac{1}{7} (nL)^6 + \frac{1}{6} (nL)^5 \left(\frac{L}{2} - nL \right) \right) \right] \\ \gamma_2 &= \left(\frac{\alpha_1}{2} \frac{L^2}{4} - \frac{\beta_1}{8} \frac{L^4}{16} + \frac{1}{144} \frac{L^6}{64} \right) - \left(\frac{\alpha_1}{2} \left(\frac{L}{2} - nL \right)^2 - \frac{\beta_1}{8} \left(\frac{L}{2} - nL \right)^4 + \frac{1}{144} \left(\frac{L}{2} - nL \right)^6 \right) \\ \beta_3 &= 1 + \frac{k+1}{nL} \left(\frac{L}{2} - nL \right)\end{aligned}$$

With a slab deflection w_m , using Eq. (7.27) one can find a value of q denoted as q_{1m}^* .

Then the contribution of membrane forces to load bearing capacity according to element 1 is given by:

$$e_{1m}^* = \frac{q_{1m}^*}{p_{y,\theta}} \quad (7.28)$$

where $p_{y,\theta}$ is the yield-line load of the slab at the evaluated temperature.

b) Element 2 (e_{2m}^)*

Taking the moments about the support of Element 2 (**Fig. 7.9**):

$$2 \int_{AB} N_y \left(\left(1 - \frac{y}{l/2} \right) w_m - w_{zy} \right) dy - \text{Moment due to } \sum q = 0 \quad (7.29)$$

Substituting Eq. (7.14) to (7.29):

$$2 \int_0^{l/2} \left[bKT_0 - \frac{y}{l/2} (k+1) bKT_0 \right] \times \left[\left(1 - \frac{y}{l/2} \right) w_m - w_{zy} \right] dy - \frac{1}{6} ql \times (nL)^2 = 0 \quad (7.30)$$

After taking the integrations and regrouping (**Appendix D**):

$$\begin{aligned} & q \left[\frac{1}{6} (nL)^2 l - 2bKT_0 \frac{1}{EI_2} (nL) \left[-\frac{61}{23040} l^5 + \frac{31}{32256} (k+1) \right] \right] \\ &= 2bKT_0 \left[\frac{w_m l}{4} - \alpha T_{,zb2} \frac{1}{24} l^3 - \frac{1}{12} (k+1) w_m l + (k+1) \alpha T_{,zb2} \frac{l^3}{64} \right] \end{aligned} \quad (7.31)$$

With a slab deflection w_m , using Eq. (7.31) one can find a value of q denoted as q_{2m}^* . Then the contribution of membrane forces to load bearing capacity according to element 2 is given by:

$$e_{2m}^* = \frac{q_{2m}^*}{p_{y,\theta}} \quad (7.32)$$

7.3.4.4 Effect of membrane forces on bending resistance (e_{1b}, e_{2b})

a) Element 1 (e_{1b})

The effect of the membrane forces on the bending resistance along the yield lines is evaluated by considering the yield condition when axial load is present, as proposed by Wood (1961). In the case of the short span the bending moment resistance in the presence of an axial force is given by:

$$\frac{M_N}{\mu M_0} = 1 + \frac{2(g_0)_1}{3 + (g_0)_1} \left(\frac{N_x}{KT_0} \right) - \frac{1 - (g_0)_1}{3 + (g_0)_1} \left(\frac{N_x}{KT_0} \right)^2 \quad (7.33)$$

where $(g_0)_1$ is the parameter defining the flexural stress block in the short span.

The factor e_{1b} does not depend on the boundary condition. Thus it is similar to the Bailey-BRE method (**Appendix C**).

$$e_{1b} = \frac{M}{\mu M_0 L} = 2n \left[1 + \frac{2(g_0)_1}{3 + (g_0)_1} \frac{b}{2} (k-1) - \frac{1 - (g_0)_1}{3 + (g_0)_1} \frac{b^2}{3} (k^2 - k + 1) \right] + (1 - 2n) \left(1 - \frac{2(g_0)_1}{3 + (g_0)_1} b - \frac{1 - (g_0)_1}{3 + (g_0)_1} b^2 \right) \quad (7.34)$$

b) Element 2 (e_{2b})

Similarly, the factor e_{2b} can be calculated by Eq. (7.35)

$$e_{2b} = \frac{M}{M_0 l} = 1 + \frac{2(g_0)_2}{3 + (g_0)_2} \frac{b}{2} K (k-1) - \frac{1 - (g_0)_2}{3 + (g_0)_2} \frac{b^2}{3} K (k^2 - k + 1) \quad (7.35)$$

where $(g_0)_2$ is the parameter defining the flexural stress block in long span.

Using Eqs. (7.3), (7.4) and (7.5), the load bearing capacity of the slab enhanced by TMA can be estimated.

7.3.5 Verification of resistance of the edge beams

The required minimum values of bending resistance for the edge beams are determined based on the two composite beam-slab collapse mechanisms as shown in **Fig. 7.3**. These collapse mechanisms were proposed by Abu *et al.* (2010) and have been included in SCI Publication P390. Their checking process is summarised as follows.

Case 1: Yield line parallel to the unprotected beam

Considering a unit displacement along the yield line, the rotation of the yield line can be calculated as follows:

$$\text{Rotation of yield line} = 2 \frac{1}{l/2} = \frac{4}{l}$$

$$\text{Internal work} = (ML_{eff} + 2M_{b,1}) \frac{4}{l} = \frac{4ML_{eff}}{l} + \frac{8M_{b,1}}{l}$$

where: L_{eff} is the effective length of the yield line discounting the effective width of the slab which acts as a T-flange of the composite edge beam; M is the resistance moment of the slab per unit length of the yield line;

$$\text{External work} = \frac{1}{2} pLl$$

Equating the internal and external works, we have:

$$\frac{4ML_{eff}}{l} + \frac{8M_{b,1}}{l} = \frac{1}{2} pLl$$

$$M_{b,1} = \frac{pLl^2 - 8ML_{eff}}{16} \quad (7.36)$$

where p is the uniformly distributed load to be supported by the floor design zone in fire condition.

Case 2: Yield line perpendicular to the unprotected beam

In this case, rotation of yield line = $2 \frac{1}{L/2} = \frac{4}{L}$

$$\text{Internal work} = (Ml_{eff} + nM_{hot} + 2M_{b,2}) \frac{4}{L} = \frac{4Ml_{eff}}{L} + \frac{8M_{b,2}}{L} + \frac{4nM_{hot}}{L}$$

where: l_{eff} is the effective length of the yield line discounting the effective width of the slab which plays as a T-flange of the composite edge beam; M is the resistance moment of the slab per unit length of the yield line; M_{hot} is the moment resistance of unprotected interior beams; n is the number of the unprotected interior beams.

$$\text{External work} = \frac{1}{2} pLl$$

Equating the internal and external works, we have:

$$\begin{aligned} \frac{4Ml_{eff}}{L} + \frac{8M_{b,2}}{L} + \frac{4nM_{hot}}{L} &= \frac{1}{2} pLl \\ \frac{4Ml_{eff}}{L} + \frac{8M_{b,2}}{L} + \frac{4nM_{hot}}{L} &= \frac{1}{2} pLl \\ M_{b,2} &= \frac{pL^2l - 8Ml_{eff} - 8nM_{hot}}{16} \end{aligned} \quad (7.37)$$

where: p is the uniformly distributed load to be supported by the floor design zone in fire condition.

7.3.6 Procedure to implement the model

The proposed model is carried out in two stages as shown in **Fig. 7.13**. The first stage is *thermal input*, at which temperature distribution in the member sections should be known, either from fire tests, numerical analyses or from analytical thermal models. The second stage, *mechanical response*, is determined through a deflection limit. Based on that deflection, the overall enhancement factor is calculated, and then the total load-bearing capacity of the slab can be determined.

The proposed enhancement factor is derived in **Sections 7.3.3** and **7.3.4**. In the last step, the moment resistance of the protected edge beams needs to be checked to ensure that the composite failure modes do not occur. The derivation of appropriate design equations for the edge beams is proposed by Abu *et al.* (2010) and described in Simms and Bake (2010). **Section 7.3.5** has presented this in detail. The implementation of this model can be achieved by using Matlab program or Excel spreadsheets.

Fig. 7.13 Procedure to calculate the total load-bearing capacity of a composite slab-beam system

The in-plane force equilibrium gives two equations in terms of two unknowns k and b . For a given slab deflection w_m , the out-of-plane force equilibrium gives two equations to calculate two enhancement factors according to Elements 1 and 2, based on which the overall enhancement factor can be determined. Detailed procedure to calculate the proposed overall enhancement factor (step 2.2) is as follows:

- (2.2.1) It is assumed that there are only two failure modes of the slab panel as shown in **Fig. 7.2**. Calculate the parameter k by Eq. (7.6) and choose the

parameter b as the minimum value of the two from Eqs. (7.7) and (7.8).

(2.2.2) Calculate the yield-line load of the slab at temperature θ .

(2.2.3) Using the slab deflection determined based on the deflection limit (**Section 8.3**), calculate the enhanced load capacity according to Element 1 by Eq. (7.27), and then determine the enhancement factor e_{1m}^* by Eq. (7.28).

(2.2.4) Calculate the enhanced load capacity according to Element 2 by Eq. (7.31), and then determine the enhancement factor e_{2m}^* by Eq. (7.32).

(2.2.5) Calculate the effect of membrane forces on bending resistance e_{1b} and e_{2b} by Eqs. (7.34) and (7.35).

(2.2.6) Use Eqs. (7.3) and (7.4) to calculate the overall enhancement factor,

In step (2.3), the load-bearing capacity of the slab enhanced by TMA ($q_{s,\theta}$) is determined from Eq. (7.5).

In step (2.4), if the slab panel has unprotected interior beams, the load-carrying capacity of the interior beams ($q_{b,\theta}$) can be added to the load-bearing capacity of the slab enhanced due to TMA ($q_{s,\theta}$). The assumption made by Bailey that these unprotected interior beams should be treated as composite is verified in **Section 7.4.3**. The total load-bearing capacity of the slab-beam system is:

$$q_{t,\theta} = q_{s,\theta} + q_{b,\theta} = e^* \times p_{y,\theta} + q_{b,\theta} \quad (7.38)$$

In step (2.5), using the load determined from step (2.4), calculate the required bending resistance of the edge beams in fire by using Eqs. (7.36) and (7.37). These values are then checked against the bending resistance of the beams in fire. If the beams have sufficient bending resistance, then the composite beam-slab collapse mechanisms shown in **Fig. 7.3** do not occur. Therefore, the assumption used in step (2.2) is validated. If the beams have failed, increase the size of the edge beams and then calculate again.

7.4 Model validation

In order to validate the proposed model against the test results, the deflections of the slabs recorded at the failure time are used.

7.4.1 Resistance of the edge beams at failure time

As discussed in Section 7.4, the first step is to check if the resistance of the edge beams satisfies the condition in Eqs. (7.36) and (7.37) so that either of the two failure modes of the slab panel (**Fig. 7.2**) may occur. The calculation of the plastic moment resistance of the composite edge beams at elevated temperatures follows the principle of EN 1994-1-2 (Clause 4.3) taking account of the degree of shear connection between the steel section and the concrete. Illustrative calculations for the secondary edge beam (PSB) of S1 is given below.

Problem statement of specimen S1:

Dimensions of the slab $L = 2.25\text{m}$; $l = 2.25\text{m}$; the overall slab thickness $h_s = 55\text{mm}$. Concrete strength $f_{ck} = 36.3\text{MPa}$. Mesh reinforcement $f_{yk} = 543\text{MPa}$ with $A_{sx} = 84.82\text{mm}^2$; $A_{sy} = 84.82\text{mm}^2$; and $d_x = 38\text{mm}$; $d_y = 35\text{mm}$. Test load is $p = 15.68\text{kN/m}^2$. The structural layout of S1 is shown in **Fig. 5.1(a)**. S1 failed after 85.6min of heating. Recorded temperatures at the slab top surface (ST), mesh (SM), and bottom surface (SB) are $ST = 172^\circ\text{C}$, $SM = 391^\circ\text{C}$, $SB = 630^\circ\text{C}$, respectively.

Temperatures of PSB at failure: top flange $PT = 763^\circ\text{C}$; beam web $PW = 744^\circ\text{C}$; bottom flange $PB = 752^\circ\text{C}$. Properties of the protected secondary edge beams: $h = 80\text{mm}$; $b_f = 80\text{mm}$; $t_f = 9.14\text{mm}$; area of the steel section $A = 2020\text{mm}^2$; $f_y = 435\text{MPa}$. Connection degree at ambient: $n_{c,20} = 1.0$.

Fig. 7.14 Assumed failure mode for the minimum moment resistance of PSB of S1

Table 7.1 Moment resistance of the protected secondary edge beam in fire – Specimen S1

Parameters	Values
Effective width of the slab (Clause 5.4.1.2 EN1994-1-1)	
- Internal side $b_{eff1} = \min(L_e / 8, b_i)$	$b_{eff1} = \min(2250 / 8; 2250 / 2) = 281.3\text{mm}$
- End support side $b_{e2} = \min(L_e / 8; 450)$	$b_{e2} = \min(2250 / 8; 450) = 281.3\text{mm}$
$b_{eff2} = b_0 + (0.55 + 0.025L_e / b_{e2})$	$b_{eff2} = 210.9\text{mm}$
Effective width : $b_{eff} = b_0 + b_{eff1} + b_{eff2}$	$b_{eff} = 0 + 281.3 + 210.9 = 492.2\text{mm}$
Reduction factor of compressive strength of concrete	$k_{c,\theta} = k_{c,\theta}(172^\circ\text{C}) = 0.964$
Reduction factor of yield strength of steel	$k_{y,\theta} = k_{y,\theta}(752^\circ\text{C}) = 0.168$
Temperature of shear stud can be assumed as: $0.8PT$ (Clause 4.3.4.2.5 EN1994-1-1)	$\theta_u = 0.8 * 763 = 610.4^\circ\text{C}$
Reduction factor for strength of shear stud	$k_{u,\theta} = k_{u,\theta}(610.4) = 0.445$
Connection degree of the beam at 20°C	$n_{c,20} = 1.0$
Connection degree of the beam in fire	

$n_{c,\theta} = \frac{n_{c,20} k_{u,\theta} \gamma_{M,v}}{k_{y,\theta} \gamma_{M,fi,v}}$	$n_{c,\theta} = \frac{1.0 \times 0.445 \times 1.25}{0.163 \times 1.0} = 3.42 > 1.0$ $\rightarrow$ Full shear connection
<p>Thickness of the slab in compression in fire</p> $x_c = \frac{A \times k_{y,\theta} f_y / \gamma_{M,fi,a}}{0.85 \times b_{eff} \times k_{c,\theta} f_{ck} / \gamma_{M,fi,c}}$	$x_c = \frac{2020 \times 0.168 \times 435 / 1.0}{0.85 \times 492.2 \times 0.964 \times 36.3 / 1.0} = 10.08 \text{ mm}$ $x_c = 10.08 \text{ mm} < h_s = 55 \text{ mm}$
<p>Bending moment resistance</p> $M_{fi,Rd,2} = \frac{A \times k_{y,\theta} f_y}{\gamma_{M,fi,a}} \left(\frac{h}{2} + h_s - \frac{x_c}{2} \right)$	$M_{fi,Rd,2} = \frac{2020 \times 0.168 \times 435}{1.0} \left(\frac{80}{2} + 55 - \frac{10.08}{2} \right) / 10^6$ $M_{fi,Rd,2} = 13.28 \text{ kNm}$
<p>Where:</p> <ul style="list-style-type: none"> - b_0 is the distance between the centres of the outstand shear connectors. There is only one row of connectors $\rightarrow b_0 = 0$. - $\gamma_{M,fi,a}, \gamma_{M,v}$ and $\gamma_{M,fi,v}$ are the partial safety factor of the steel section, the shear stud at ambient and elevated temperatures, respectively. - $\gamma_{M,fi,c}$ is the partial safety factor of concrete in fire. 	

The required bending moment resistance of PSB can be calculated by Eq. (7.35).

$$M_{b,2} = \frac{pL^2l - 8M_{eff}l - 8nM_{hot}}{16}$$

where:

M is the resistance moment of the slab per unit length of the yield line

$$x_c = \frac{543 \times 84.82}{0.85 \times 36.3 \times 1000} = 1.49 \text{ mm}$$

$$M = 543 \times 84.82 \left(35 - \frac{1.49}{2} \right) / 10^6 = 1.58 \text{ kNm}$$

l_{eff} is the effective length of the yield line

$$l_{eff} = 2250 - 2 \times 492.2 = 1265.6 \text{ mm}$$

$$p = 15.68 \text{ kN/m}^2$$

$$\rightarrow M_{b,2} = \frac{15.68 \times 2.25^2 \times 2.25 - 8 \times 1.58 \times 1.2656}{16} = 10.11 \text{ kNm}$$

It is found that $M_{fi,Rd,2} = 13.28\text{kNm} > M_{b,2} = 10.11\text{kNm}$, therefore the composite failure mode shown in **Fig. 7.14** did not occur.

Similarly, the resistance of the edge beams at failure time of all specimens can be calculated and summarised in **Table 7.2**.

Table 7.2 Verification of resistance of the edge beams

<i>Test</i>	<i>P_{test}</i> <i>kN/m²</i>	-----Temperatures-----						<i>M_{b,1}</i> by Eq. (7.36)	<i>M_{fi,Rd,1}</i>	<i>M_{b,2}</i> by Eq. (7.37)	<i>M_{fi,Rd,2}</i>	Composite failure modes?
		<i>MT</i>	<i>MW</i>	<i>MB</i>	<i>PT</i>	<i>PW</i>	<i>PB</i>	<i>kNm</i>	<i>kNm</i>	<i>kNm</i>	<i>kNm</i>	
<i>S1</i>	15.6	668	735	756	763	744	752	9.7	19.8	10.1	13.3	No
<i>S3-FR</i>	16.0	402	504	486	428	508	502	9.9	79.7	10.4	54.0	No
<i>S2-FR-IB</i>	15.1	760	865	870	762	854	879	9.3	9.4	8.3	9.0	No
<i>P215-M1099</i>	15.6	600	689	740	638	731	691	9.5	22.6	4.5	22.7	No
<i>P368-M1099</i>	15.3	726	760	777	675	735	764	9.4	17.7	4.7	17.6	No
<i>P486-M1099</i>	15.5	717	785	771	744	796	803	9.5	18.1	5.5	9.3	No
<i>P215-M1356</i>	15.4	738	804	824	724	783	775	9.3	13.1	7.0	13.5	No
<i>P215-M2110</i>	15.6	766	821	785	765	806	782	9.1	24.8	6.1	12.7	No

It can be seen in **Table 7.2** that the composite failure modes did not occurred. This conforms to the experimental observations.

7.4.2 Specimens without interior beams

Among the eight specimens, there were two without interior beams, i.e. S1 and S3-FR. At the loading phase, the two specimens were loaded to a value of 15.8kN/m^2 , corresponding to a load ratio of about 2.0 for S1 and S3-FR, respectively. The tests were terminated when reinforcement fractured above the edge beams (S1) or compression ring crushed near the corners of the slab (S3-FR). It should be noted that at the time of ‘failure’, the slab could not resist any more load. Illustrative calculation for S1 can be found in **Appendix E**. **Table 7.3** summarises the results of the specimens without the interior beams.

Table 7.3 Validation of the proposed model against the specimens without interior beams

<i>Test</i>	<i>p_{test}</i> <i>kN/m²</i>	---Temperature---			<i>w_m</i> <i>mm</i>	<i>p_{y,θ}</i> <i>kN/m²</i>	<i>Predicted</i>	<i>Prediction/Test</i>
		<i>ST</i> °C	<i>Mesh</i> °C	<i>SB</i> °C			<i>load</i> <i>kN/m²</i>	
<i>S1</i>	15.6	172	391	630	131	7.80	13.5	0.86
<i>S3-FR</i>	16.0	60	150	351	115	8.14	14.7	0.92

$$M = \mathbf{0.89}$$

As can be seen in **Table 7.3**, the proposed model gives a conservative result for the specimens without interior beams. The prediction/test values are 0.86 and 0.92 for S1 and S3-FR, respectively.

7.4.3 Specimens with unprotected interior beams

Among the eight specimens, six had two unprotected interior beams. At failure, the load of 15.8kN/m² would be supported by the slab together with the residual bending moment resistance of the unprotected interior beams. Therefore, further steps are needed to validate the model against the specimens with the interior beams as follows:

- Check if the interior beams have failed yet using the bending moment capacity method in BS EN 1994-1-2.
- If the interior beams had failed, the beams would form plastic hinges and the load-carrying mechanism would change to a two-way system (load from slab panel -> supporting edge beams -> columns). As a result, the load of 15.8kN/m² would be totally resisted by the slab. The total enhancement factor is calculated and compared with the enhancement factor determined from the test.
- If the beams had not failed yet, the test load of 15.8kN/m² was resisted by both the slab and the beams. The load carried by the slab is the test load minus part of the load supported by the interior beams. This way the test enhancement factor can be compared to the prediction.

When calculating part of the load supported by the unprotected interior beams, Bailey proposed that the interior beams can be considered as composite beams. However, by doing so, it counts the contribution of part of the slab above the beams twice. Therefore, to verify whether these beams should be considered as a composite or a steel beam, the author conducted two cases as follows. Case 1 treats the unprotected interior beam as a composite beam, whereas Case 2 assumes the unprotected interior beam as a steel beam.

7.4.3.1 Unprotected interior beams are treated as composite beams (Case 1)

Table 7.4 shows the results for Case 1, where $M_{Rd,\theta}$ is the bending moment capacity of interior composite beam. This value is compared with the design bending moment of the beam at fire condition ($M_{Ed,\theta}$) to determine whether the beam has failed or not. It should be noted that these specimens had two interior beams, and both are taken into account in the calculation.

Table 7.4 Validation of the model with the specimens with interior beams – Case 1:
The unprotected interior beam is treated as a composite beam

Test	Temp. USB	Temperature of slab ($^{\circ}\text{C}$)			w_m mm	$M_{Ed,\theta}$ kNm	$M_{Rd,\theta}$ kNm	USB	$q_{b,\theta}$ kN/m ²	$q_{s,\theta}$ kN/m ²	$q_{t,\theta}$ kN/m ²	p_{test}	
		ST	M	SB									
S2-FR-IB	892	95	512	664	177	7.2	5.8	failed	0.0	11.7	11.7	15.1	0.77
P215-M1099	761	108	348	602	124	7.4	14.7	not failed	20.7	15.0	35.7	15.6	-
P368-M1099	814	118	316	688	118	7.2	10.0	not failed	14.1	14.4	28.4	15.3	-
P486-M1099	819	164	478	748	139	7.5	9.4	not failed	13.3	15.7	28.9	15.8	-
P215-M1356	842	101	351	734	121	7.3	12.2	not failed	12.2	14.9	27.1	15.4	-
P215-M2110	777	228	508	715	143	7.4	13.2	not failed	18.6	14.6	33.2	15.6	-

where w_m is the maximum slab deflection recorded in the tests; $M_{Ed,\theta}$ is design bending moment of the interior beam in fire; $M_{Rd,\theta}$ is bending moment resistance of the composite interior beam in fire; $q_{b,\theta}$ is the increase in the load capacity due to residual bending moment resistance of the unprotected interior beams; $q_{s,\theta}$ is the

enhanced slab capacity due to TMA; $q_{t,\theta}$ is the predicted total capacity of the slab; p_{test} is the test load.

It can be seen in **Table 7.4** that in six specimens, only the prediction for S2-FR-IB is conservative with the prediction/test value is 0.77. Therefore, it can be concluded if the unprotected interior beams are treated as composite beams, the prediction of the total capacity of the beam-slab systems is not conservative. Therefore, the interior beams did not behave as composite sections under fire conditions.

7.4.3.2 Unprotected interior beams are treated as steel beams (Case 2)

The calculation results for Case 2 in which *the bending moment capacity of interior beam is calculated as a steel beam* are shown in **Table 7.5**. It should be noted that the enhanced slab capacity ($q_{s,\theta}$) due to TMA is unchanged in these two cases, because this value is calculated based on the lower-bound yield-line mechanism, assuming that the interior beams have zero resistance (the 3rd assumption).

Table 7.5 Validation of the model with the specimens with interior beams – Case 2:
The unprotected interior beam is treated as a steel beam

Test	Temp.	Temperature of slab (°C)			w_m mm	$M_{Ed,\theta}$ kNm	$M_{Rd,\theta}$ kNm	USB	$q_{b,\theta}$ kN/m ²	$q_{s,\theta}$ kN/m ²	$q_{t,\theta}$ kN/m ²	p_{test}	$q_{t,\theta}/p_{test}$
		USB	ST	M									
S2-FR-IB	892	95	512	664	177	7.2	1.9	failed	0	11.7	11.7	15.1	0.77
P215-M1099	761	108	348	602	124	7.4	4.9	failed	0	15.0	15.0	15.6	0.96
P368-M1099	814	118	316	688	118	7.2	3.2	failed	0	14.4	14.4	15.3	0.94
P486-M1099	819	164	478	748	139	7.5	3.1	failed	0	15.7	15.7	15.8	0.99
P215-M1356	842	101	351	734	121	7.3	2.8	failed	0	14.9	14.9	15.4	0.96
P215-M2110	777	228	508	715	143	7.4	4.3	failed	0	14.6	14.6	15.6	0.94

$$M = 0.93$$

Table 7.5 indicates that the proposed model gives conservative and good predictions for the specimens with interior beams if the latter are treated as bare steel beams under fire conditions. For S2-FR-IB, a great discrepancy of 23% was found. This

was possibly because S2-FR-IB failed at a mesh temperature of 512°C where there was significant reduction in strength and stiffness. In spite of this, the average discrepancy between the predictions and the test results is only 7% including S2-FR-IB.

7.4.4 Verification of deflection profile of the edge beams at elevated temperatures

Comparisons between the predictions of the deflection profile of the edge beams by using Eqs. (7.17) and (7.22) and the test results are conducted. A similar trend is obtained for all specimens. **Fig. 7.15** shows the representative results for S1 (series I) and P215-M1099 (series II).

a) S1

b) P215-M1099

Fig. 7.15 Comparisons of deflection profiles of the edge beams – S1 and P215-M1099

It can be seen that Eqs. (7.17) and (7.22) give reasonably good and conservative predictions for the deflections of the edge beams. The discrepancy between the predictions and the test results may be attributed by the 4th assumption where the protected edge beams are assumed to be simply supported without any horizontal restraint. In the tests, although the beam-to-column connections were designed as nominal ‘pinned’ connections, there was some degree of lateral restraint from thermal expansion of the protected edge beams. This restraint caused greater deflections compared to the predictions.

7.5 Comparisons with the Bailey-BRE model

In this section, the predictions of the load-bearing capacity of the slab from the proposed model and from the Bailey-BRE model are compared and discussed. For the specimens without interior beams (S1 and S3-FR), the comparison is very straightforward. For the specimens with interior beams, the comparison is also conducted with two cases. Case 1 treats the unprotected interior beam as a composite beam, whereas Case 2 assumes the unprotected interior beam as a steel beam. The calculation principles are similar as discussed in **Section 7.4.3**.

Since the Bailey-BRE method cannot consider deflection of the edge beams, the enhancement factor predicted by the Bailey-BRE method is calculated with two deflections as shown in Eqs. (7.39) and (7.40).

$$\text{Absolute deflection: } w_m \quad (7.39)$$

$$\text{Relative slab deflection: } w_r = w_m - \frac{1}{4}(w_{MB1} + w_{MB2} + w_{PSB1} + w_{PSB2}) \quad (7.40)$$

where w_m is the maximum slab deflection at failure; w_{MB1} , w_{MB2} are the deflection of two main edge beams corresponding to w_m ; w_{PSB1} , w_{PSB2} are the deflection of two secondary edge beams corresponding to w_m .

Table 7.6 Comparisons of the proposed and Bailey-BRE models for the specimens without interior beams

<i>Test</i>	<i>p_{test}</i>	<i>MB Defl.</i>	<i>PSB Defl.</i>	<i>Slab Defl.</i>	<i>Total capacity</i>			<i>Prediction / Test</i>		
					<i>Bailey (Eq. 7.39)</i>	<i>Bailey (Eq. 7.40)</i>	<i>Proposed model</i>	<i>Bailey (Eq. 7.39)</i>	<i>Bailey (Eq. 7.40)</i>	<i>Proposed model</i>
	<i>kN/m²</i>	<i>mm</i>	<i>mm</i>	<i>mm</i>	<i>kN/m²</i>	<i>kN/m²</i>	<i>kN/m²</i>			
<i>S1</i>	15.6	28	58	131	15.8	12.91	13.5	1.01	0.83	0.86
<i>S3-FR</i>	16.0	33	28	115	17.1	14.6	14.7	1.07	0.91	0.92
<i>M =</i>								1.04	0.87	0.89

Table 7.7 Comparisons of the proposed and Bailey-BRE models for the specimens with interior beams – Case 1: The unprotected interior beam is treated as a composite beam

Test	p_{test}	MB defl.	PSB defl.	Slab defl.	Rela tive defl.	Total capacity (kN/m ²)			Prediction / Test		
						Bailey (Eq. 7.39)	Bailey (Eq. 7.40)	Proposed model	Bailey (Eq. 7.39)	Bailey (Eq. 7.40)	Proposed model
						kN/m ²	mm	mm	mm	mm	kN/m ²
S2-FR- IB	15.1	56	84	177	107	14.0	10.4	11.7	0.92	0.69	0.77
P215- M1099	15.6	38	57	124	76	38.7	34.9	35.7	2.48	2.23	2.28
P368- M1099	15.3	51	83	118	51	31.4	26.0	28.4	2.05	1.70	1.86
P486- M1099	15.5	55	94	139	64	31.9	26.1	28.9	2.06	1.68	1.86
P215- M1356	15.4	59	88	121	48	30.3	24.5	27.1	1.97	1.59	1.76
P215- M2110	15.6	39	79	143	84	35.2	31.7	33.2	2.26	2.03	2.13
M =									1.96	1.66	1.78

Table 7.8 Comparisons of the proposed and Bailey-BRE models for the specimens with interior beams – Case 2: The unprotected interior beam is treated as a steel beam

Test	P_{test}	MB defl.	PSB defl.	Slab defl.	Rela tive defl.	Total capacity			Prediction / Test		
						Bailey (Eq. 7.39)	Bailey (Eq. 7.40)	Proposed model	Bailey (Eq. 7.39)	Bailey (Eq. 7.40)	Proposed model
	kN/m^2	mm	mm	mm	mm	kN/m^2	kN/m^2	kN/m^2			
S2- FR-IB	15.1	56	84	177	107	14.0	10.4	11.7	0.92	0.69	0.77
P215- M1099	15.6	38	57	124	76	18.0	14.2	15.0	1.15	0.91	0.96
P368- M1099	15.3	51	83	118	51	17.3	11.9	14.4	1.13	0.78	0.94
P486- M1099	15.5	55	94	139	64	18.7	12.9	15.7	1.20	0.83	1.01
P215- M1356	15.4	59	88	121	48	18.1	12.2	14.9	1.17	0.79	0.97
P215- M2110	15.6	39	79	143	84	16.6	13.1	14.6	1.07	0.84	0.94
$M =$									1.11	0.81	0.93

From the comparisons between the two models against the test results, the following points can be drawn:

- (i) For the specimens without interior beams, **Table 7.6** indicates that if applying the absolute slab deflection (Eq. (7.39)), the Bailey-BRE method over-predicts the test results by about 4%, whereas the method is conservative if applying the relative slab deflection (Eq. (7.40)). The proposed method predicts the test results more accurately than the Bailey-BRE method, because it can take into account the deflection shape of the edge beams.
- (ii) For the specimens with two interior beams, if the unprotected interior beams are treated as a composite beam, **Table 7.7** indicates that both the Bailey-BRE and the proposed approaches give un-conservative predictions. Therefore, it can be concluded that the unprotected interior beams should not be treated as composite beams.
- (iii) **Table 7.8** shows that when the interior beams are treated as steel beams, if applying the absolute slab deflection, i.e. the deflection of the edge beams is not taken into account, the results predicted by the Bailey-BRE method

are greater than the test results by about 11% in average. If applying the relative deflection (Eq. (7.40)), the Bailey-BRE method is conservative.

- (iv) The proposed model gives fairly accurate predictions of the load-bearing capacity of composite beam-slab systems compared to the test results. The discrepancy between the proposed model and the test results is 11% (**Table 7.6**) for the specimens without interior beams, and only 7% (**Table 7.8**) for the specimens with interior beams; while the discrepancy between the Bailey-BRE model and the test results is 13% for the specimens without interior beams, and 19% for the specimens with interior beams.

7.6 Conclusions

In this chapter, limitations of the Bailey-BRE method have been discussed. Based on that, the author proposes a semi-analytical model based on the Bailey-BRE method which takes into account the vertical deflections of the edge beams. The model gives more accurate predictions compared to the Bailey-BRE method. The proposed model considers the beam-slab interaction under fire conditions. It also gives information of the edge beams, which the Bailey-BRE method cannot provide.

The comparisons between the two approaches, i.e. the proposed and the Bailey-BRE models, against the test results indicate that the deflection of the fire-protected edge beams should be considered when calculating the load-bearing capacity of the composite beam-slab systems. This can be done by using the Bailey-BRE method with *the relative deflection* of the slab, or by using the proposed model.

When calculating the contributions of unprotected interior beams, the interior beams should be treated as steel beams. This reflects a more accurate load-transfer mechanism at tensile membrane stage as follows: slab $\rightarrow$ supporting edge beams $\rightarrow$ columns. By doing so, the prediction of the total load-bearing capacity of the composite beam-slab systems is conservative.

In the next chapter, from the experimental, numerical and analytical analyses conducted, recommendations in terms of design point of view are introduced and discussed.

CHAPTER 8 DESIGN RECOMMENDATIONS

8.1 Introduction

Based on a number of experimental, numerical and analytical studies on TMA of composite floor slabs at elevated temperatures as discussed in **Chapter 2**, a design guide (Newman *et al.* 2006) has been released in UK, *SCI Publication P288*, in which the BRE-Bailey method (Bailey and Moore 2000a; Bailey 2001a; Bailey 2003; 2004) was adopted. Thereafter, an updated user guide of P288, *SCI Publication P390* (Simms and Bake 2010), has been released.

SCI Publication P288 “Fire Safety Design: A new approach to multi-storey buildings” (Newman *et al.* 2006) (referred hereafter as P288) sets out the design philosophy for the Bailey-BRE method, while SCI Publication P390 “TSLAB v3.0 User Guidance and Engineering Update” (Simms and Bake 2010) (referred hereafter as P390) provides information on the operations of TSLAB software v3.0. This software tool has been developed by SCI to facilitate the Bailey-BRE method. Both P288 and P390 must be used together when applying the Bailey-BRE method to design steel-framed buildings with composite slabs. Engineers may consider using TSLAB v3.0 software to facilitate the design. Many buildings in the UK have since benefited from the application of the method, resulting in reduced fire protection costs (Newman *et al.* 2006).

SCI P390 has been updated to Eurocode requirements. It consists of new engineering verification for the edge beam capacity and the modified mesh temperature. With the latest release of P390, a number of Sections in P288 are no longer applicable. They include:

- Part 2 Section 1 of P390 which provides recommendations for structural design of the floor panels. This section is intended to complement the information given in Section 3 Part A of P288, and also provides a

replacement of Section 3.1.5 which covers the effect of additional load on boundary beams.

- Part 2 Section 2 of P390 which provides information on the design of fire-protected beams on the perimeter of each floor design zone. This section complements the basic information given in Section 3.2 Part A of P288.

This chapter aims to provide commentary and clarifications for SCI P288 and P390 with regard to Singapore design context. Therefore, Singapore National Annex (SS EN) is used as a reference for this purpose. This chapter includes discussions on temperature distribution of structural members (**Section 8.2**), deflection limit (**Section 8.3**), design procedure (**Section 8.4**) and recommendations for structural elements (**Section 8.5**). Conclusions are given in **Section 8.5**.

It is important to note that SCI P288 and P390 and the design commentary shall not be used to replace SS EN 1994-1-1 (2004) and SS EN 1994-1-2 (2005). They only provide design guidance on an advanced method based on the performance-based approach that engineers may use in the design of composite steel-framed buildings subjected to fire conditions in order to reduce fire-protection costs for interior secondary beams.

8.2 Temperature distribution of structural members

8.2.1 Fire exposure

P288 stipulated that the Bailey-BRE method may be applied to buildings in which structural elements are assumed to be exposed to the standard temperature-time curve or parametric temperature-time curve, both as defined in EN 1991-1-2 (2002). However, the simple design method was developed mainly on the basis of full scale natural fire tests in which the floors were subjected to fully developed compartment fires. Therefore, the first question here is whether the design method could be applied to the fire design based on the standard temperature-time curve (ISO 834 fire curve).

This question has been answered by a research project titled “Fire resistance assessment of partially protected composite floors (FRACOF)” conducted in France in 2008 (Zhao *et al.* 2008). The FRACOF fire test was intended to provide experimental evidence on the behaviour of composite slab-beam floor systems exposed to the ISO 834 fire curve and to widen the application of the simple design method in France. The test results showed that the whole floor remained structurally robust under a long duration (120 minutes as expected), despite fracture of steel mesh reinforcement in the concrete slab. After that, a design guide named “FRACOF design guide” was released. Therefore, it can be concluded that the simple design method can be applied to the ISO 834 fire curve.

Advanced models, *e.g.* computational fluid dynamic models, may also be used to define a temperature-time curve for a natural fire scenario. The resulting temperature-time curve can be used as the input data for the design method. It is noteworthy that SCI recommends that only readers familiar with fire safety engineering should attempt to use any information based on parametric fire curves.

In all cases, the normal provisions of national regulations regarding the means of escape should be followed.

8.2.2 Composite slabs

The temperature distribution in a composite slab can be determined using semi-analytical models or heat transfer models taking into account the exact shape of the slab and adhering to Clause 4.4.2 of EN 1994-1-2.

Thermal response of composite slabs determined from semi-analytical models

The advantage of using semi-analytical models is that the exact shape of the slab, *i.e.* shape of profile sheeting, can be taken into account. Engineers can use a semi-analytical model in TLAB V3.0 which uses a 2D finite difference heat transfer method (presented P390 Appendix A) with confidence. This is because this method has been used for many years by SCI to predict temperature distributions in steel and steel-concrete composite cross sections and has been shown to give reasonably

accurate predictions of thermal responses of sections in fire resistance tests.

Another semi-analytical model is the method stipulated in EN 1994-1-2 Annex D. However, the Building and Construction Standards Committee responsible for SS EN 1994-1-2 (2009) does not recommend the use of Annex D. This is because it has been shown that the method given in EN 1994-1-2 Annex D would be too conservative to be useful for profiled sheeting shapes containing large stiffeners or indentations (Francis and Simms 2012). SS NA advised that there is an alternative to Annex D on the website www.steel-ncci.co.uk. This alternative method suggests that temperatures of the strips required for plastic analysis of the section can be calculated directly by a series of thickness-temperature relationships in equation form. A slab thickness-temperature relationship is also provided to calculate the temperature of reinforcement within the ribs of the slab.

Besides, the Eurocode Non Contradictory Complementary Information (NCCI) for fire resistance design of composite slabs (Francis and Simms 2012) can be used to determine thermal responses of unprotected composite slabs subjected to the standard fire.

Thermal response of composite slabs based on finite element (FE) models

As an alternative, thermal responses of unprotected composite slabs subjected to standard or natural fires can be determined from FE models. Since they have been validated and shown to give accurate predictions of thermal distributions in composite slabs, the FE model proposed in **Section 3.4.4** can be used to predict thermal responses of composite slabs. It should be noted that those validation exercises are based on the assumption of using an average depth of solid slab for both thermal and structural analyses. Although the model has been validated well with the Fracof test, this particular assumption deserves more discussions.

Fig. 8.1 shows the thermal responses using different slab thicknesses. Clearly, using the exact shape of composite slabs would give the most accurate prediction. However, this causes difficulties in modelling. On the other hand, the approach of

using thin continuous slab thickness provides the most conservative results among the three methods (full, average or thin continuous slab thickness). Therefore, Bailey (2001b) examined the ‘weighted average’ reinforcement temperature approach which uses a weighted average of the reinforcement temperatures through the ribs and thinner parts on structural response of the slab. He suggested that higher reinforcement temperatures in the thinner part of the slab should be used with an average depth for structural calculation as a conservative approach.

Fig. 8.1 Schematic of thermal response of composite slabs

Abu *et al.* (2007) questioned on the validity of using ‘weighted average’ reinforcement temperatures. They conducted two numerical analyses with one using the average depth (**Fig. 8.1(b)**), and the other applying thin continuous depth (**Fig. 8.1(c)**) to study effect of reinforcement temperature on the structural behaviour of the slab. They found that these two approaches give similar results at tensile membrane stage.

Therefore, once again it can be concluded that the assumption of using an average solid slab thickness for both thermal and structural analyses is reasonable.

8.2.3 Composite beams

Temperature distributions of unprotected interior beams or protected edge beams can be determined based on the simple method given in SS EN 1994-1-2, Section 4.3.4.2 as presented below.

The increase of temperature $\Delta\theta_{a,t}$ of various parts of *an unprotected steel beam* during the time interval Δt may be determined from Eq. (8.1).

$$\Delta\theta_{a,t} = k_{shadow} \left(\frac{1}{c_a \rho_a} \right) \left(\frac{A_i}{V_i} \right) \dot{h}_{net} \Delta t \quad (8.1)$$

where k_{shadow} is a correction factor for the shadow effect; c_a is the specific heat of steel; ρ_a is the density of steel; A_i/V_i is the section factor for part i of the cross-section; $\dot{h}_{net}$ is the design value of the net heat flux per unit area.

The increase of temperature $\Delta\theta_{a,t}$ of various parts of *a protected steel beam* during the time interval Δt may be determined from Eq. (8.2).

$$\Delta\theta_{a,t} = \left[\left(\frac{\lambda_p / d_p}{c_a \rho_a} \right) \left(\frac{A_{p,i}}{V_i} \right) \left(\frac{1}{1 + w/3} \right) (\theta_t - \theta_{a,t}) \Delta t \right] - \left[(e^{w/10} - 1) \Delta\theta_t \right] \quad (8.2)$$

with $w = \left(\frac{c_p \rho_p}{c_a \rho_a} \right) d_p \left(\frac{A_{p,i}}{V_i} \right)$

where λ_p is the thermal conductivity of the fire protection material; d_p is the thickness of the fire protection material; $A_{p,i}$ is the area of the inner surface of the fire protection material per unit length; c_p is the specific heat of the fire protection material; ρ_p is the density of the fire protection material; θ_t is the ambient gas temperature at time t ; $\Delta\theta_t$ is the increase of the ambient gas temperature during the time interval Δt .

Alternatively, the proposed FE model presented in **Section 3.4.4.2** can be used to determine the temperature distribution of an unprotected or insulated composite beam.

8.3 Deflection limit

Although the Bailey-BRE method and the proposed approach predict reasonably well the ultimate load of the composite floor system, both cannot predict the failure point of the system. Therefore, a deflection limit has to be assumed, and then the enhancement above the yield-line load bearing capacity of the slab due to tensile membrane action is calculated based on that limiting deflection. Eq. (8.3) shows the deflection limit proposed by Bailey and Moore (2000a). This deflection limit is estimated by combining the components due to thermal curvature and strain in the reinforcement.

$$w = \frac{\alpha(T_2 - T_1)l^2}{19.2h} + \sqrt{\left(\frac{0.5f_y}{E_a}\right)_{\text{Reinf } t_{20^\circ\text{C}}}} \frac{3L^2}{8} < w_{\max} = \frac{\alpha(T_2 - T_1)l^2}{19.2h} + \frac{l}{30} \quad (8.3)$$

where α is the coefficient of thermal expansion (12×10^{-6} for normal weight concrete; T_2 and T_1 are the bottom and top surface temperatures of the slab, respectively; h is the average depth of the concrete slab; l and L are the shorter and longer spans of the slab panel; f_y and E_a are the yield strength and the elastic modulus of the reinforcing steel at ambient temperature.

Based on the Cardington fire test, Bailey proposed that the term $(T_2 - T_1)$ is assumed to be 770°C for fire exposure below 90 minutes and 900°C thereafter. This value was obtained from the test data from the Cardington fire tests (Bailey *et al.* 1999). In SCI P390 (Simms and Bake 2010), the deflection limit is also calculated by Eq. (8.3), except that the term $(T_2 - T_1)$ is based on the temperature calculated at the bottom and top surfaces of the slab at each time step.

Table 8.1 shows that the deflection limits proposed by Bailey and Moore (2000a) and SCI P390 (Simms and Bake 2010) are smaller than the maximum deflection recorded in the tests. It should be noted that in Series I and II, failure of the slab did occur, and all the tests were terminated when fracture of reinforcement had been identified. Therefore, it can be concluded that both deflection limits proposed by

Bailey and SCI P390 are conservative. Using the proposed model with these deflection limits gives conservative predictions.

Table 8.1 Comparison of the deflection limit proposed by the Bailey-BRE method and SCI P390

<i>Test</i>	<i>Slab depth</i>	<i>Deflection due to thermal curvature</i>	<i>Deflection due to mechanical strain</i>	<i>Bailey deflection limit (Eq. (8.3))</i>	<i>SCI P390 deflection limit</i>	<i>Max. deflection in test*</i>	<i>Bailey deflection limit / Test deflection</i>	<i>TSLAB deflection limit / Test deflection</i>
	<i>mm</i>	<i>mm</i>	<i>mm</i>	<i>mm</i>	<i>mm</i>	<i>mm</i>		
<i>S1</i>	55	44	54	98	80	131	0.75	0.61
<i>S2-FR-IB</i>	55	44	54	98	86	177	0.55	0.49
<i>S3-FR</i>	58	42	57	99	73	115	0.86	0.63
<i>P215-M1099</i>	57	43	57	99	84	124	0.80	0.68
<i>P368-M1099</i>	58	42	57	99	88	118	0.84	0.74
<i>P486-M1099</i>	55	52	57	108	90	139	0.78	0.65
<i>P215-M1356</i>	58	42	57	99	91	121	0.81	0.75
<i>P215-M2110</i>	59	48	57	105	83	143	0.74	0.58
<i>M =</i>							0.77	0.64

* Test terminated when fracture of reinforcement had been identified.

8.4 Design procedure

Based on the comparisons of the proposed model and the Bailey-BRE method against the tests results, the design procedure to predict the load-bearing capacity of a composite slab-beam floor system (Bailey and Moore 2000a) was revised. It should be noted that the procedure proposed by Bailey has been adopted in SCI P390. **Fig. 8.2** shows the revised design procedure in which three following points should be noted:

- (i) The allowable deflection of slab can be estimated by Eq. (8.3). This is *the absolute deflection*. When using the model proposed in this thesis to predict the load-bearing capacity of the floor system, this deflection can be used directly since the proposed model has already taken into account of the deflections of the edge beams.

- (ii) If using the Bailey-BRE method, the allowable deflection should be the deflection of the slab relative to those of the edge beams. Based on the relative deflection of the slab to estimate the load-bearing capacity of the floor system, one can obtain conservative predictions.
- (iii) When calculating the contribution of the load-bearing capacity of the slab from unprotected interior beams under fire conditions, as discussed in **Section 7.5**, the interior beams should be treated as steel beams.

Fig. 8.2 Revised design procedure based on Bailey and Moore (2000)

8.5 Recommendations for structural elements

8.5.1 Reinforcement

SCI P288 stipulates that the yield strength and ductility of the reinforcing steel material should be specified in accordance with the requirements of BS EN 10080, in which the characteristic yield strength of reinforcement is between 400 and 600MPa. In order that the reinforcement has sufficient ductility to allow the development of tensile membrane action, Class B or Class C reinforcement should be specified.

One important criterion in order that TMA can be mobilised in composite slabs is the ductility of reinforcing steel. As discussed in **Section 7.3.2**, failure of the slab could occur due to fracture of the mesh across the slab short span. If the elongation of reinforcing steel is too large, reinforcement fracture can also occur across the protected edge beams, which may lead to a loss of integrity and insulation performance of the floor before load bearing failure is reached. On the other hand, if the mesh reinforcement does not have sufficient elongation capacity, the reinforcement may fracture before the required deflection of the slab is reached, which may limit the mobilisation of tensile membrane action.

Therefore, the question arises about the criterion to be applied to the elongation capacity of reinforcing steel. EN 1992-1-2 implies that for plastic design the minimum elongation capacity at ultimate stress for reinforcing steel must be at least 5%. This question has been investigated by Vassart and Zhao (2010). They conducted an extensive parametric study on elongation of reinforcement. The parameters varied were the load magnitude, size of the slab panel, fire duration of 30, 60, 90, 120min. They found that the maximum elongation of reinforcing steel mesh for all investigated floors was lower than 5% for all the fire durations. Therefore, it can be concluded that an elongation capacity of reinforcing steel of 5% can be used as a minimum requirement for the mobilisation of TMA.

8.5.2 Protected edge beams

In the design of using TMA concept, the floor slab is divided into several slab panels. The edge beams which bound each slab panel must be designed to achieve a period of fire resistance required by the floor slab. This will ensure that the pattern of yield lines and the associated enhancement due to TMA actually happen in practice.

The required moment of resistance of the edge beams is calculated by considering alternative yield line patterns that would allow the slab to fold along an axis of symmetry without developing TMA. Details of the calculation method are given in **Section 7.4.4**. This procedure shall be followed.

On the other hand, when TMA is mobilised, the secondary and main fire-protected edge beams carry more load at the fire limit state, compared with the other limit states. Therefore, the load ratios of the beams increase. This must be considered when calculating the required moment capacity of these beams to ensure that they provide sufficient bending resistance to allow development of TMA in the slab. A critical temperature for the beams can then be calculated and appropriate levels of fire protection can be applied to ensure that this critical temperature is not exceeded during the required fire resistance.

8.5.3 Connections

Types of connections

SCI P288 stipulates that ‘simple’ joints such as those with flexible end plates, fin plates and web cleats shall be used when adopting the simple design method.

The steel frame building tested at Cardington had flexible end plate and fin plate connections. Partial and full failures of some of the joints were observed during the cooling phase of the Cardington fire tests; however, these failures did not result in any collapse of the floor slab.

In the case where the plate was torn off the end of the beam, no collapse occurred because the floor slab transferred the shear to the other supporting beams. This highlights the important role of the composite floor slab, which can be achieved with proper lapping of mesh reinforcement in the slab.

The resistance of simple joints should be verified using the rules given in EN 1993-1-8 (2005c). To ensure that a simple joint does not transfer significant bending moment, it must have sufficient rotation capacity, which can be achieved by detailing the joint such that it meets geometrical limits. Guidance on geometrical limits and initial sizing to ensure sufficient rotation capacity of the joint is given in Access-steel documents SN013 and SN016 (Nunez 2005a; 2005b).

Fig. 8.3 'Simple' beam-to-column connections

Fire protection of connections

In cases where both structural elements to be connected are fire protected, the protection appropriate to each element should be applied to the parts of the plates or angles in contact with that element. If only one element requires fire protection, the plates or angles in contact with the unprotected elements may be left unprotected.

8.5.4 Columns

In order to mobilise TMA in composite slab-beam floor systems, the supporting columns of the slab panel must have sufficient resistance. Therefore, steel columns are required to be protected for the full height. Design of the protected columns should in accordance with EN 1993-1-2 and EN 1994-1-2.

8.6 Conclusions

This chapter provides validation of the deflection limit proposed by Bailey and Moore (2000) against the test results. The comparison shows that this deflection limit is conservative and therefore can be used in estimating the enhanced load-bearing capacity of the slab due to TMA. Thereafter, some special aspects of determination of thermal responses of composite slabs, unprotected interior beams, and protected edge beams were highlighted.

One noteworthy point is that unprotected interior beams should be treated as steel beams when calculating the contribution of the interior beams on the total load-bearing capacity of the floor system. This reflects a more accurate load-transfer mechanism at tensile membrane stage as follows: slab → supporting edge beams → columns. This is a new author's recommendation compared to SCI P390 where the unprotected interior beams are treated as *composite beams*.

More importantly, the edge beams which bound each slab panel must be designed to achieve a period of fire resistance required by the floor slab. Therefore, after the ambient design, the resistance of the edge beams should be checked against alternative yield line patterns that would allow the slab to fold along an axis of symmetry without developing TMA.

Tensile membrane action is definitely an inherent capacity of composite floor systems under fire conditions. However, in order that TMA can be mobilised in the systems without compromising the safety, special stipulations for structural members shall be followed closely in the design process.

CHAPTER 9 CONCLUSIONS AND FUTURE WORKS

9.1 Conclusions

This research programme investigated the structural behaviour of composite beam-slab floor systems (CB-S systems) under fire conditions. The research presented in this thesis achieved its stated objectives, including: (i) to obtain a physical understanding of the structural behaviour of CB-S systems under fire conditions based on experimental investigation; (ii) to develop finite element models which are able to predict structural and thermal behaviour of CB-S systems in fire; (iii) to propose a semi-analytical model to estimate the load-bearing capacity of CB-S systems in fire taking into account of deflection of the supporting edge beams on TMA; (iv) to suggest design recommendations on the current design guides using tensile membrane action (TMA) concept, SCI Publications P288 and P390.

(i) *The experimental programme*

In this programme, eight composite beam-slab floor systems were tested to failure under fire conditions. *Series I* included three specimens with the aim to study the effects of unprotected secondary beams and rotational edge restraint on the behaviour of floor slabs. *Series II* included five specimens in order to investigate the effect of bending stiffness of the protected main and secondary edge beams on the behaviour of floor slabs. The test setup was very reliable because the sum of internal forces calculated from the data of strain gauges on supporting steel columns were close to the measured external loads as verified in Series I. Furthermore, temperature distributions of the slabs were shown to only vary across the slab thickness and were in double symmetry.

The experimental observations are of significance, and the key points are summarized as follows. *Firstly*, it is observed that TMA was mobilised in all the specimens. Therefore, it is reliable to apply TMA under fire conditions to enhance the load-bearing capacity of CB-S systems. *Secondly*, reinforcement fracture above

the protected edge beams, rather than fracture of tension reinforcement in the central region, was the final collapse mode of the CB-S systems. *Thirdly*, it is possible to omit fire protection for interior secondary beams, which leads to cost savings of the fire-protection materials without compromising the safety of the systems under fire conditions.

Significant findings from Series I

- (1) Reinforcement continuity over the supporting edge beams and the presence of interior beams can reduce deflection of the slab and greatly enhance its load-bearing capacity.
- (2) The presence of interior beams significantly affects the magnitude and distribution of stresses in the mesh reinforcement. The maximum tensile stresses did not necessarily occur at the slab centre, but were located in the concrete slab above the protected edge beams.
- (3) Rotational restraint induces intense stress concentration above the edge beams, which can result in premature concrete crushing at the four corners of the slab and wide cracks over the protected edge beams.

Significant findings from Series II

- (1) As the stiffness of the protected edge beams increased (main or secondary edge beams), the slab deflection decreased. However, the experimental results showed that 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, this beneficial effect was lost.
- (2) The composite slab-beam systems with unprotected interior beams can mobilise TMA in fire. However, no clear tensile zone at the slab centre can be found. Across the section which is perpendicular to the secondary beams, there are only compressive forces in the slab. This is because parts of the slab act as a T-flange for the composite secondary beam, which balance the tensile forces in the steel beams.

Similarities between the test results of two test series

- (1) The tests revealed different ‘composite’ failure modes of the floor assemblies. In all specimens, severe cracks appeared in the vicinity of the protected edge beams, resulting in fracture of reinforcement at these regions. In the case without unprotected interior beams (S3-FR test), failure occurred through crushing of compression ring at the corners of the slab. No fracture of reinforcement at the mid-span of the slabs was observed.
- (2) TMA was mobilised at a deflection equal to 0.9 to 1.0 of the slab thickness irrespective of the presence of interior beams or bending stiffness of the edge beams. The imminence 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 the slab edges; and (c) horizontal and vertical displacements of the columns. In all the tests, these indicators were very consistent in terms of occurrence times.
- (3) None of the connections had failed or fractured during the heating or the cooling phase. Therefore, it can be concluded that if the beam-to-beam and beam-to-column connections were designed accurately according to EN 1993-1-8, the connections have sufficient robustness so that TMA can be mobilised in the slab.

(ii) Numerical modelling of the slab action under fire conditions

Using the results and observations from the experiment, finite element models using ABAQUS/Explicit have been developed. The numerical models were validated extensively with the author’s tests and with small- and full- scale tests in literature. The models gave accurate predictions of the structural behaviour of composite slab-beam systems subjected to fire.

It is found that the accuracy of a FE model in predicting the structural/thermal behaviour of composite beam-slab systems at large deflections is controlled by the accuracy of temperature predictions at reinforcing mesh.

(iii) The proposed semi-analytical model

A semi-analytical model was developed to estimate the enhancement of the load-bearing capacity of composite slab-beam systems under fire conditions due to TMA, which can consider the vertical deflections of edge beams. The model has been shown to give conservative and more accurate predictions compared to the Bailey-BRE method. It gives information of the edge beams which the Bailey-BRE method does not provide.

The comparisons between the two approaches, *i.e.* the proposed and the Bailey-BRE models, against the test results indicate that the deflection of the fire-protected edge beams should be taken into consideration when calculating the load-bearing capacity of the CB-S systems. This can be done by either using the Bailey-BRE method with *the relative deflection* of the slab, or by using the proposed model with a more accurate result obtained.

(iv) Design recommendations

Tensile membrane action is definitely an inherent capacity of composite floor systems under fire conditions. However, in order that TMA can be mobilised in the systems without compromising the safety, special stipulations for structural members shall be followed closely in the design process.

One important point different from the Bailey-BRE method or SCI P390 is that unprotected interior beams should be treated as steel beams when calculating the contribution of the interior beams on the total load-bearing capacity of the floor system.

More importantly, the edge beams which bound each slab panel must be designed to achieve a period of fire resistance required by the floor slab. This will ensure that the pattern of yield lines and the associated enhancement due to TMA actually happen in practice. Therefore, after the beams have been designed for ambient temperature, the moment of resistance of the edge beams should be checked against the alternative

yield line patterns that would allow the slab to fold along an axis of symmetry without developing TMA.

9.2 Future works

Experimental study of composite floor systems enhanced by new materials, such as Engineered cementitious composite (ECC) or Fiber-reinforced concrete (FRC)

This experiment showed that the concrete slabs developed full-depth cracks leading to reinforcement fracture above the edge beams. This suggests that if the slabs were to be made of ECC or FRC, tensile membrane action may develop further. However, using ECC or FRC may lead to other types of failure modes under fire conditions, such as severe spalling since ECC and FRC are more condensed than normal concrete. Therefore, an experimental programme should be conducted for this topic.

Experimental investigation with different detailing rules to enhance the load-bearing capacity of composite floor systems under fire conditions

In this experiment, reinforcement was fractured and full-depth cracks appeared in the vicinity of the protected edge beams, leading to failure of the slabs due to loss of integrity “E”. If we can find the solutions to delay this type of failure, more tensile membrane forces may be mobilised and the load-bearing capacity of the floor assemblies can be enhanced. Therefore, future experimental study is needed, which focuses on detailing rules to enhance the load-bearing capacity of the slabs at tensile membrane state.

Numerical study on full-scale composite floor systems under fire conditions

A weakness of current experimental studies on composite floor systems in laboratory condition is that the specimens are not full scale. On the other hand, full scale fire tests of composite steel-framed buildings are too costly. In such a situation, numerical simulation is an alternative solution. The proposed numerical models in this research, which has been validated with the current test results, can be used to model full scale composite buildings at large deformations. However, design guideline should specify failure criteria for engineers.

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APPENDIX A

Design of Prototype Slab Panel

APPENDIX A1

Design of composite slab to EN1994-1-1

1. Data

Floor plan data

Span of slab /	$l = 3.00$	m
Span type	triple	
Beam width	300	mm

Properties of profiled sheeting

Use the re-entrant composite floor deck

Depth of decking	$h_p = 51$	mm	
Characteristic yield strength	$f_{yk,p} = 350$	N/mm ²	; $\gamma_{M0} = 1$
Design thickness	$t_p = 0.96$	mm	
Effective area of cross-section	$A_p = 1759$	mm ² /m	
Height to neutral axis	$e = 16.73$	mm	
Second moment of area of steel core	$I_p = 6.21E+05$	mm ⁴ /m	
Design value of modulus of elasticity	$E_p = 2.10E+05$	N/mm ²	
Plastic moment of resistance	$M_{pa,Rk} = 5.29$	kNm/m	; $\gamma_{M0} = 1$
Sagging bending resistance	$M_{a,Rk}^+ = 6.34$	kNm/m	; $\gamma_{M0} = 1$
Hogging bending resistance	$M_{a,Rk}^- = 7.93$	kNm/m	; $\gamma_{M0} = 1$
Characteristic resistance to vertical shear	$V_{pa,Rk} = 60$	kNm/m	
Characteristic resistance to longitudinal shear	$\tau_{u,Rk} = 0.306$	N/mm ²	; $\gamma_{Vs} = 1.25$
$m = 184$ N/mm ²	$k = 0.0530$	N/mm ²	

Properties of materials

Concrete

Concrete grade	C30 /37		
Type of concrete	Normal Weight Concrete		
Concrete wet density	$\rho_{wc} = 2400$	kg/m ³	
Concrete dry density	$\rho_c = 2350$	kg/m ³	
Design value of compressive strength	$f_{cd} = f_{ck} / \gamma_c = 20.00$	N/mm ²	; $\gamma_c = 1.5$
Secant modulus of elasticity	$E_{cm} = 3.30E+04$	N/mm ²	

Reinforcement

Characteristic yield strength	$f_{yk} = 500$	N/mm ²	
Design yield strength	$f_{yd} = f_{yk} / \gamma_s = 435$	N/mm ²	; $\gamma_s = 1.15$
Design value of modulus of elasticity	$E_s = 2.00E+05$	N/mm ²	

Slab data

Overall slab depth $h_t = 150$ mm
Slab depth above steel decking $h_c = 99$ mm
Effective depth $d_p = h_t - e = 133.27$ mm
Volumn of concrete

$$v_c = 0.141 \quad \text{m}^3/\text{m}^2$$

2. Loadings per unit area of composite slab

Loadings per unit area of composite slab for each stage are shown in Table 1.

Table 1 Loadings per unit area of composite slab

Type of load	Denote & Formular	aracteristic actions (kN/m ²)	Partial factors of safety	
			ULS*	SLS**
<u>Construction stage</u>				
Self weight of the sheeting		$g_p = 0.14$	1.35	1.0
Self weight of wet concrete	$g_c = \rho_{wc} \times 9.81 \times v_c =$	3.32	1.35	1.0
Distributed construction load		$q_1 = 2.00$	1.5	
Concentrated construction load		$q_2 = 0.00$	1.5	
Total		7.67	(factored load used for ULS)	
		3.46	(factored load used for SLS)	
<u>Composite stage</u>				
Self weight of slab	$g_1 = \rho_c \times 9.81 \times v_c + g_p =$	3.39	1.35	1.0
Partitions, floor finishes and services		1.70	1.35	1.0
Imposed load (for office)		$q = 3.00$	1.50	
Total		11.37	(factored load used for ULS)	
<u>For deflection of composite slab</u> (frequent combination)				
		$\sum \gamma_G g_i + \psi_1 q =$	6.59	
			$\psi_1 = 0.5$ (offices)	

*ULS: ultimate limit state; **SLS: serviceability limit state

3. Verification of the sheeting as shuttering

3.1. Ultimate limit state

Calculate the internal forces

The unpropped sheeting will be verified as a two-span continuous beam resisted uniform loads.

Effective span l_e : $l_e = \min(l, \text{clear span} + \text{deck depth}) = 2.75$ m

Permanent actions: $1.35 \times (g_p + g_c) \times 1 = 4.67$ kN/m

Due to variable actions: $1.5 \times (q_1 + q_2) \times 1 = 3.00$ kN/m

Load cases

Permanent actions	M_{1a}	M_{2a}	M_B	M_C	R_A	R_B
2.83	0.88	-3.53	-3.53	5.14	14.13
Variable actions	M_{1a}	M_{2a}	M_B	M_C	R_A	R_B
1.82	0.57	-2.27	-2.27	3.30	9.08
2.30	-1.14	-1.14	-1.14	3.71
1.70	-1.14	-1.14	-0.41
-2.65	-0.76		-9.90
0.38	-1.51		
Envelopes	5.13	2.59	-6.18	-5.81	8.85	23.21

	$M_{1a} \text{ (kNm)}$	$M_{1a} \text{ (kNm)}$	M_B	M_C	R_A	R_B
Envelopes	5.13	2.59	-6.18	-5.81	8.85	23.21

Internal force envelopes

Maximum sagging bending moment:	$M_{Ed}^+ = 5.13$	kNm/m
Maximum hogging bending moment:	$M_{Ed}^- = 6.18$	kNm/m
Maximum reaction	$V_{Ed} = 23.21$	kN/m

Design check

- Positive bending:	$M_{Rd}^+ = \frac{M_{a,Rk}^+}{\gamma_{M0}} = \frac{M_{a,Rk}^+}{1.0} = 6.34 \text{ kNm/m}$	$>$	$M_{Ed}^+ = 5.13 \text{ kNm/m}$	OK
- Negative bending	$M_{Rd}^- = \frac{M_{a,Rk}^-}{\gamma_{M0}} = \frac{M_{a,Rk}^-}{1.0} = 7.93 \text{ kNm/m}$	$>$	$M_{Ed}^- = 6.18 \text{ kNm/m}$	OK
- Support reaction	$V_{pa,Rd} = \frac{V_{pa,Rk}}{\gamma_{M1}} = \frac{V_{pa,Rk}}{1.0} = 60.00 \text{ kN/m}$	$>$	$V_{Ed} = 23.21 \text{ kN/m}$	OK

- Iteration, moment and support reaction: Check at first interior support position

$$M_{Ed}^- / M_{Rd}^- + V_{Ed} / V_{pa,Rd} = 1.17 \leq 1.25 \quad \text{OK}$$

As the sagging moment does not exceed the design resistance, a 20% redistribution of hogging moment is taken based on equilibrium of the continuous decking. The hogging moment after redistribution is:

$$M_{Ed1}^- = 0.8 M_{Ed}^- = 4.95 \text{ kNm/m}$$

Recheck the sagging moment: $M_{Ed1}^+ = 6.363 > 6.34 \text{ kNm/m}$ **Recalculate**

Recheck the iteration condition:

$$\frac{M_{Ed1}^-}{M_{Rd}^-} + \frac{V_{Ed}}{V_{pa,Rd}} = 1.01 < 1.25 \quad \text{OK}$$

All design checks are OK at the ultimate limit state.

3.2. Serviceability limit state

Assume that the section of the sheeting is fully effective.

$$\delta = \frac{(2.65g_p + 3.4g_c)l_o^4}{384E_s I_p} = 14.0 \text{ mm}$$

Allowable deflection is:

$$\delta_{\max} = \min(l_o / 180 ; 20 \text{ mm} ; \text{slab depth} / 10) = 15.0 \text{ mm} \quad \text{OK}$$

The serviceability limit state of the construction state is verified.

4. Verification of the composite slab

4.1. Ultimate limit state

Total loads (factored load used for ULS): $w = 11.37$ kN/m²

Effective span: $L_e = \min(l, \text{clear span} + h_t) = 2.85$ m

The continuous slab will be designed as a series of simply supported spans.

The mid-span bending moment is: $M_{Ed} = wL_e^2/8 = 11.55$ kNm/m

The design vertical shear is: $V_{Ed} = wL_e/2 = 16.21$ kN/m

Bending resistance check

There must be full shear connection, so that the design compressive force in the concrete, $N_{c,f}$, is equal to the yield force for the steel.

$$N_{c,f} = A_p f_{yp,d} = 615650.00 \text{ N/m}$$

The depth of the stress block, for full shear connection, is:

$$x_{pl} = \frac{N_{c,f}}{0.85 f_{cd} b} = 36.21 \text{ mm} < h_c = 99.00 \text{ mm}$$

Therefore, the plastic neutral axis is above the sheeting. The distribution of longitudinal bending stresses is shown in Figure 1.

Figure 1 Cross-section of composite slab, and stress blocks for sagging bending

The design resistance to sagging bending is, for full shear connection:

$$M_{Rd} = N_{c,f} (d_p - 0.5 x_{pl}) = 70.90 \text{ kNm/m} > M_{Ed} = 11.55 \text{ kNm/m} \quad \text{OK}$$

Vertical shear resistance check

The recommended value for minimum value (v_{min}) is:

$$v_{min} = 0.035 \left[1 + (200/d_p)^{1/2} \right]^{3/2} f_{ck}^{1/2} = 0.44 \text{ N/mm}^2$$

in which d_p taken as not less than 200. In this problem, d_p is taken **200 mm**

The design resistance to vertical shear is:

$$V_{Rd} = (b_0/b) d_p v_{min} = 43.53 \text{ kN/m} > V_{Ed} = 16.21 \text{ kN/m} \quad \text{OK}$$

with b is the pitch of deck ribs taken as 152.50 mm

b_0 is the effective width of the concrete ribs, taken as 112.50 mm

d_p is the depth to the centroidal axis, taken as 133.27 mm

(b_0 : for re-entrant profiles, the minimum width should be used)

Longitudinal shear check

For longitudinal shear, it is assumed that there is no end anchorage.

m-k method

The design shear resistance is:

$$V_{i,Rd} = bd_p \left[m A_p / (b L_s) + k \right] / \gamma_{vs} = 51.66 \quad \text{kN/m}$$

The value used are:

$$\begin{aligned} b &= 1.00 \text{ m} & d_p &= 133.27 \text{ mm} \\ L_s &= l/4 = 0.75 \text{ m} & A_p &= 1759 \text{ mm}^2 \\ m &= 184 \text{ N/mm}^2 & k &= 0.053 \text{ N/mm}^2 \end{aligned}$$

This value must not be exceeded by the vertical shear in the slab. Therefore, $V_{i,Rd}$ is taken as:

$$V_{i,Rd} = \min(V_{i,Rd}, V_{Rd}) = 43.53 \text{ kN/m} > V_{Ed} = 16.21 \text{ kN/m} \quad \text{OK}$$

4.2. Serviceability limit state

Control of cracking of concrete

As the slab is designed as simply supported, only anti-crack reinforcement is needed. The cross-sectional area of the reinforcement above the ribs should be not less than 0.2% for un-propped construction.

$$\min A_s = 0.002 \times b \times h_c = 198 \quad \text{mm}^2/\text{m}$$

Use A252 with $A_s = 2.52 \text{ cm}^2/\text{m}$ ($\phi 8 \text{ a200}$)

Deflection

The total load is taken as, using the frequent combination for calculating the deflection of composite slab:

$$w = \sum \gamma_{G_i} g_i + \psi_1 q = 6.59 \quad \text{kN/m}^2$$

For the calculation of deflection, firstly, the slab is considered to be simply supported.

The following approximations apply:

- the second moment of area may be taken as the average of the values for the cracked and un-cracked section;
- for concrete, an average value of the modular ratio, n , for both long and short-term effects may be used.

The short-term elastic moduli and modular ratios are:

$$n_0 = E_p / E_{cm} = 6.36$$

For buildings, EN 1994 permits the simplification that all strains may be assumed to be twice their short-term value. This is done by using modular ratios:

$$n = 2n_0 = 12.73$$

Second moment of area for the cracked section in sagging region

The depth of neutral axis (counted from the top of slab):

$$x_c = \frac{n A_p}{b} \left(\sqrt{1 + \frac{2 b d_p}{n A_p}} - 1 \right) = 58.04 \quad \text{mm}$$

The second moment of area for the cracked section is:

$$I_{\infty} = \frac{b x_c^3}{12n} + \frac{b x_c (x_c / 2)^2}{n} + A_p (d_p - x_c)^2 + I_p = 1.57 \text{E}+07 \quad \text{mm}^4/\text{m}$$

Second moment of area for the un-cracked section

The depth of neutral axis (counted from the top of slab):

$$x_u = \frac{\sum A_i z_i}{\sum A_i} = \frac{bh_c^2/2 + b_o h_p (h_t - h_p/2) + n A_p d_p}{bh_c + b_o h_p + n A_p} = 67.64 \text{ mm}$$

The second moment of area for the un-cracked section is:

$$I_{cu} = \frac{bh_c^3}{12n} + \frac{bh_c (x_u - h_c/2)^2}{n} + \frac{b_o h_p^3}{12n} + \frac{b_o h_p}{n} \left(h_t - x_u - \frac{h_p}{2} \right)^2 + A_p (d_p - x_u)^2 + I_p$$

with	$h_c = 99 \text{ mm}$	$h_p = 51 \text{ mm}$
	$b_o = 112.50 \text{ mm}$	$h_t = 150 \text{ mm}$
	$d_p = 133.27 \text{ mm}$	$A_p = 1759 \text{ mm}^2$

Therefore, $I_{cu} = 1.87\text{E}+07 \text{ mm}^4/\text{m}$

Average I_c of the cracked and un-cracked section

$$I_c = \frac{I_{cc} + I_{cu}}{2} = 1.72\text{E}+07 \text{ mm}^4/\text{m}$$

The total deflection is:

$$\delta = \frac{5}{384} \frac{w L_e^4}{E_s I_c} = 1.65 \text{ mm}$$

The allowable deflection is:

$$[\delta] = 1/250 L_e = 11.4 \text{ mm} \quad > \quad \delta = 1.65 \text{ mm} \quad \text{OK}$$

All design checks are OK at both the ultimate limit state and the serviceability limit state.

APPENDIX B

Drawings of test specimens and test set-up

GERNERAL NOTES

1. MATERIALS

- (1) All welds are required along the entire length unless indicated otherwise.
- (2) All bolts and nuts are in grade 8.8.
- (3) Hole sides for bolts are made 2mm larger in diameter than the nominal size of bolts unless indicated otherwise.

: Fillet weld on both side of joint with thickness t

: Fillet weld on the arrow-side of joint with thickness t

: Fillet weld on the other-side of joint opposite the arrow-side with thickness t

- (4) All specimens use materials with the properties as follows:

Structural steel: grade S355

Reinforcement: characteristic yield strength of 500 N/mm²

Concrete class C30/37: f_{ck} = 30 MPa; f_{cu} = 37 MPa

- (5) Shear stud

Diameter x Connector height: 13 x 40 (mm)

Steel grade: S275

2. FIRE PROTECTION

- (1) Columns C1, Joints, MB1, MB2, MB3, MB4, PSB1, PSB2: protected in fire
USB: Unprotected in fire; Composite slab: Unprotected in fire
- (2) The members as indicated above and in dwgs. are protected in fire using intumescent paint. The thickness of the paint layer is determined by Contractor in order that the protected member temperature is limited to less than 650°C at 60 minutes.
- (3) The thickness and material of intumescent paint layer shall be obtained approval from NTU before hand.

3. FOR CONTRACTOR

- (1) All dimensions are in milimeter (mm) except height level in meter (m)
- (2) Before commencing work, Contractor needs to check carefully all the dimensions of specimens in drawings. If there is any conflict, Contractor should inform immediately to the designer.
- (3) Any modification or correction shall be approved by NTU beforehand.

BILL OF MATERIAL

SPECIMEN	DENOTE	Q'TY	SHAPE	LENGTH (m)	WEIGHT (kg)	REM.
S1	PSB1	02	Built-up 80x80x15.68	2.25		
	MB1	02	W 130x130x28.1	2.25		
	C1	04	UC 152x152x30	1.25		
S2-FR-IB	PSB1	02	Built-up 80x80x15.68	2.25		
	USB	02	Built-up 80x80x15.68	2.25		
	MB1	02	W 130x130x28.1	2.25		
	C1	04	UC 152x152x30	1.25		

NOTES:

- Columns C1, Joints, PSB1, PSB2, MB1, MB2: Protected in fire using intumescent paint (see general notes)
- USB: Unprotected beam
- Composite slab: Unprotected in fire
- Bill material here only shows general descriptions. Detail calculations shall be based on in detailed drawings thereafter.

DRAWING TITLE	
STRUCTURAL LAYOUT S1 & S2-FR-IB	
DRAWING NO: S-01	14Dec10 - v1
DESIGNER: NGUYEN TUAN TRUNG	

S1: SLAB REINFORCEMENT

SECTION A-A

SECTION B-B

S1: CONNECTOR ARRANGEMENT

HEADED STUD

SHEAR CONNECTOR

NOTES:

- See General Notes
- Reinforcing bars are located at midheight of concrete slab.
- Shear stud
- Diameter x Connector height: 13 x 40 (mm)
- Steel grade: S275
- Shear studs can be formed by cutting down from normal shear stud 13 x 70 (mm)

DRAWING TITLE		
SLAB DETAILS		
S1		
DRAWING NO:	S-02	Mar 11 - v3
DESIGNER:	NGUYEN TUAN TRUNG	

S2-FR-IB: SLAB REINFORCEMENT

SECTION A-A

SECTION B-B

S2-FR-IB: CONNECTOR ARRANGEMENT

HEADED STUD

SHEAR CONNECTOR

NOTES:

- See General Notes
- Reinforcing bars are located at midheight of concrete slab.
- Shear stud
- Diameter x Connector height: 13 x 40 (mm)
- Steel grade: S275
- Shear studs can be formed by cutting down from normal shear stud 13 x 70 (mm)

DRAWING TITLE		
SLAB DETAILS S2-FR-IB		
DRAWING NO:	S-03	Mar 11 - v3
DESIGNER:	NGUYEN TUAN TRUNG	

BEAM MB1

SECTION 1-1 (MB1)

Joists 127x114x29

SECTION 2-2

Joists 76x76x15

HEADED STUD

SHEAR CONNECTOR

BEAM PSB1

BEAM USB

NOTES:

- See General Notes
- Reinforcing bars are located at midheight of concrete slab.

– Shear stud
Diameter x Connector height: 13 x 40 (mm)

Steel grade: S275

Shear studs can be formed by cutting down from normal shear stud 13 x 70 (mm)

DRAWING TITLE	
<h1 style="margin: 0;">BEAM DETAILS</h1> <h2 style="margin: 0;">MB1, PSB1, USB</h2>	
DRAWING NO: S-04	Jan11 - v2
DESIGNER: NGUYEN TUAN TRUNG	

VIEW A(B) - JOINT 1

VIEW C - JOINT 1

JOINT 1

SECTION 1-1

SECTION 2-2

DRAWING TITLE	
COLUMN & CONNECTION DETAILS	
DRAWING NO: S-05	Dec10 - v1
DESIGNER:	NGUYEN TUAN TRUNG

BILL OF MATERIAL				
ELEMENT: COLUMNS & CONNECTIONS (unit: mm)				
No	SHAPE & DIMENSION	THK/LENGTH	QUANTITY/UNIT	TOTAL QUANTITY
01	-140x130x8	8	04	08
02	-120x100x6	6	04	08
03	-100x70x6	6	04	04
04	-300x300x15	15	04	08
05	UC 152x152x30	1250	04	08
06	-100x100x10	10	08	16
07	BOLT ϕ 18	L=60	16	32
08	BOLT ϕ 16	L=50	16	32
	BOLT ϕ 16	L=50	16	16

DRAWING TITLE	
COLUMN & CONNECTION DETAILS	
DRAWING NO: S-06	Dec10 - v1
DESIGNER:	NGUYEN TUAN TRUNG

PLAN VIEW

SECTION 1-1

DETAIL OF LOADING POINTS

SECTION 2-2

PERIPHERAL SURFACE

2 -200x200x15 (04 UNITS)

DRAWING TITLE	
LOADING SYSTEM	
DRAWING NO: TS-02	Nov 10 - v1
DESIGNER: NGUYEN TUAN TRUNG

SECTION 1-1

SECTION 2-2

BILL OF MATERIAL				
ELEMENT: CIRCULAR COLUMNS (4 UNITS)				
No	SHAPE & DIMENSION	THK/LENGTH	QUANTITY/UNIT	TOTAL QUANTITY
01	-300x300x15	15	01	04
02	-200x200x15	15	01	04
03	-120x200x15	15	01	04
04	CHS D114X6.3	L=395	01	04
05	BOLT ϕ 20	L=80	09	36
06	BOLT ϕ 25	L=100	02	08

NOTES:

- Circular columns are protected in fire as specimens

DRAWING TITLE	
DETAILS OF CIRCULAR COLUMN	
DRAWING NO: TS-04	Dec10 - v1
DESIGNER:	NGUYEN TUAN TRUNG

CONCRETE CYLINDERS

- CL1-A is concrete cylinder for material test at ambient temperature
- CL2-E is concrete cylinder for material test at elevated temperatures
- Numbers of concrete cylinder for both specimens are:
12 cylinder CL1-A
20 cylinder CL2-E

STRUCTURAL STEEL COUPONS

- CP1-A is coupon for material test at ambient temperature
- CP2-E is coupon for material test at elevated temperatures
- Number of coupons CP1-A for each type of beams used is 04 including:
02 coupons from beam flanges
02 coupons from beam web
- Number of coupons CP2-E for each type of beams used is 08 including:
04 coupons from beam flanges
04 coupons from beam web
- Thickness of coupons equals to the thickness of beam flange or beam web

REINFORCING REBARS

- Reinforcing samples are taken in the same batch with reinforcement used for concrete slab.
- Total numbers of RB1 for two specimens are 10

DRAWING TITLE	
MATERIAL SAMPLES	
DRAWING NO: M-01	Jan11 - v1
DESIGNER:	NGUYEN TUAN TRUNG

S3-FR

P215-M1099

P486-M1099

P215-M2110

BILL OF MATERIAL

SPECIMEN	DENOTE	Q'TY	SHAPE	LENGTH (m)	WEIGHT (kg)
S3-FR	PSB1	02	Built-up 80x80x15.68	2.25	
	MB1	02	W 130x130x28.1	2.25	
	C1	04	UC 152x152x30	1.25	
P215-M1099	PSB1	02	Built-up 80x80x15.68	2.25	
	USB	02	Built-up 80x80x15.68	2.25	
	MB1	02	W 130x130x28.1	2.25	
	C1	04	UC 152x152x30	1.25	
	PL	04	3170x55x8	3.17	
P486-M1099	PSB2	02	Joists 102x102x23	2.25	
	USB	02	Built-up 80x80x15.68	2.25	
	MB1	02	W 130x130x28.1	2.25	
	C1	04	UC 152x152x30	1.25	
	PL	04	3170x55x8	3.17	
P215-M2110	PSB1	02	Built-up 80x80x15.68	2.25	
	USB	02	Built-up 80x80x15.68	2.25	
	MB2	02	UB 203x102x23	2.25	
	C1	04	UC 152x152x30	1.25	
	PL	04	3170x55x8	3.17	

NOTES:

- Columns C1, Joints, PSB1, PSB2, MB1, MB2: Protected in fire using intumescent point (see general notes)
- USB: Unprotected beam
- Composite slab: Unprotected in fire
- Bill material here only shows general descriptions. Detail calculations shall be based on in detailed drawings thereafter.
- Details of attachment of steel plate PL will be found in separate drawings

DRAWING TITLE

STRUCTURAL LAYOUT
S3-FR & SERIES II

DRAWING NO: S-01

v2 2011-12

DESIGNER:

NGUYEN TUAN TRUNG

P368-M1099

P215-M1356

BILL OF MATERIAL

SPECIMEN	DENOTE	Q'TY	SHAPE	LENGTH (m)	WEIGHT (kg)
P368-M1099	PSB3	02	Built-up 100x80x15.32	2.25	
	USB	02	Built-up 80x80x15.68	2.25	
	MB1	02	W 130x130x28.1	2.25	
	C1	04	UC 152x152x30	1.25	
	PL	04	3170x55x8	3.17	
P215-M1356	PSB1	02	Built-up 80x80x15.68	2.25	
	USB	02	Built-up 80x80x15.68	2.25	
	MB3	02	UB 178x102x19	2.25	
	C1	04	UC 152x152x30	1.25	
	PL	04	3170x55x8	3.17	

NOTES:

- Columns C1, Joints, PSB1, PSB2, PSB3, MB1, MB2, MB3: Protected in fire using intumescent point (see general notes)
- USB: Unprotected beam
- Composite slab: Unprotected in fire
- Bill material here only shows gernal descriptions. Detail calculations shall be based on in detailed drawings thereafter.
- Details of attachment of steel plate PL will be found in separate drawings

DRAWING TITLE	
STRUCTURAL LAYOUT SERIES II	
DRAWING NO: S-02	v2 2011-12
DESIGNER: NGUYEN TUAN TRUNG	

Structural drawing of a rectangular slab with dimensions and reinforcement details. The overall dimensions are 3150 mm by 3150 mm. The drawing shows a grid of reinforcement bars with labels for additional reinforcing bars, welded joints, and slab reinforcement welded to a steel plate (PL). Dimensions are provided for various sections and bar spacing.

Dimensions and Reinforcement Details:

- Overall dimensions: 3150 mm by 3150 mm.
- Grid lines: 1, 2 (horizontal) and A, B (vertical).
- Reinforcement bars: 46 ϕ 3 @ 70 (top and bottom), 8 ϕ 3 (additional reinforcing bars).
- Welded joints: Bolt M25, 50 mm.
- Slab Reinf. welded to PL: -3150x55x5 Steel Plate (PL).
- Dimensions for sections: 450, 750, 2250, 450 (horizontal) and 450, 750, 750, 750 (vertical).

[illegible]

Figure 10.10 shows the reinforcement details of a square column. The column has a total width of 3150 mm and a total height of 3150 mm. The reinforcement is provided in four layers: 6 bars in the top and bottom layers, and 9 bars in the side layers. The column is supported by a steel plate (PL) at the base. The reinforcement details are shown for both the top and bottom views of the column.

- See General Notes for more details
- Reinforcing bars are located at midheight of concrete slab.
- Shear stud

Diameter x Connector height: 13 x 40 (mm)

Steel grade: S275

Shear studs can be formed by cutting down from normal shear stud 13 x 70 (mm)

DRAWING TITLE	
SLAB DETAILS SERIES I	
DRAWING NO : S-03	v2 2011-12
DESIGNER: NGUYEN TUAN TRUNG	

W 130x130x28.1

UB 203x102x23

[illegible]

UB 178x102x19

101.2

177.8

162

7.9

7.9

4.8

48.2

4.8

48.2

Diagram of a built-up I-section. The overall height is 100. The flange thickness is 8. The web height is 84. The flange width is 80. The web thickness is 8. The distance from the centerline of the web to the edge of the flange is 36. A note indicates a 6mm gap between the flange and the web.

Diagram showing a cross-section of a rectangular column with a circular hole. The total height is 40, the hole diameter is 19, and the hole is positioned 13 units from the bottom.

Build-up 80x80x15.68

Technical drawing of a built-up section. The drawing shows a cross-section of a built-up I-beam. The total height is 80. The flange thickness is 9. The web height is 62. The web thickness is 9. The distance from the centerline of the web to the outer edge of the flange is 6. A note is present: "Note".

[illegible]

- See General Notes for more details
- Reinforcing bars are located at midheight of concrete slab.
- Shear stud

Diameter x Connector height: 13 x 40 (mm)

Steel grade: S275

Shear studs can be formed by cutting down from normal shear stud 13 x 70 (mm)

DRAWING TITLE	
BEAM DETAILS MB3, PSB3 & USB	
DRAWING NO: B-02	v2 2011-12
DESIGNER: NGUYEN TUAN TRUNG	

[illegible]

Technical drawing of the connection between a column and a beam. The column is labeled '3' and '-100x70x8'. The beam is labeled 'Beam MB2' and 'Beam USB'. The connection is secured with '4 M16' bolts. Dimensions include a column width of 101.8, a bolt spacing of 80, and a beam width of 100. Section lines A-A and B-B are indicated at the bottom, and C-C is indicated on the right.

The diagram illustrates the cross-section of a reinforced concrete slab. Key features include:

- Reinforcement Bars:**
 - Bar 6: Top longitudinal bar, size -100x100x10.
 - Bar 9: Bottom longitudinal bars, size 6M20.
 - Bar 1b: Side longitudinal bars, size -140x180x10.
- Dimensions:**
 - Total height: 232 mm.
 - Slab thickness: 180 mm.
 - Top flange width: 100 mm.
 - Bottom flange width: 140 mm.
 - Vertical spacing between reinforcement layers: 40 mm, 150 mm, 40 mm.
 - Horizontal spacing between reinforcement bars: 30 mm, 40 mm, 40 mm, 30 mm.
- Levels and Markings:**
 - +1.155: Level of the top reinforcement layer.
 - BOTTOM OF CON. SLAB: Reference level for the bottom of the concrete slab.
 - +1.10: Level of the bottom reinforcement layer.
- Other Details:**
 - A circular hole with diameter $\phi 30$ is located in the top flange.
 - A note symbol points to the right side of the section.

Cross-section diagram of a reinforced concrete slab showing reinforcement details. The diagram includes labels for "BOTTOM OF CON. SLAB" at +1.10, "PSB3" (prestressing bed), "4M16" (top reinforcement), "-120x100x8" (bottom reinforcement), and dimensions for slab thickness (120), reinforcement spacing (50, 40, 30), and clearances (20, 40, 40, 20). A note symbol is also present.

DESIGNER: **NGUYEN TUAN TRUNG**

BOLT POSITIONS CAST TOGETHER WITH SLABS

BEAM B1
(Qu'ty: 04)

SECTION 4-4

SECTION 5-5

DETAILS OF INPLANE SYSTEM

SECTION 1-1

SECTION 2-2

SECTION 3-3

NOTES:

- Refer to drawings of the slabs details of steel plate
- Steel grade: S355

BILL OF MATERIAL				
No	SHAPE & DIMENSION	THK/LENGTH	QUANTITY/UNIT	TOTAL QUANTITY
01	RHS 160x100x6	L=3150	04	04
02	BOLT M25 8.8	L=250	20	120
03	BOLT M25 8.8	L=320	20	20

DRAWING TITLE	
INPLANE RESTRAINED BEAMS	
DRAWING NO: ad-02	2011-12-22
DESIGNER: NGUYEN TUAN TRUNG	

CONCRETE CYLINDERS

- (1) CL1-A is concrete cylinder for material test at ambient temperature.
CL2-E is concrete cylinder for material test at elevated temperatures.
(2) 6 specimens are cast into 2 batches.
For each concrete batch, numbers of concrete cylinder required are:
06 cylinder CL1-A ; 16 cylinder CL2-E

STRUCTURAL STEEL COUPONS

- (1) CP1-A is coupon for material test at ambient temperature
CP2-E is coupon for material test at elevated temperatures
(2) Number of coupons required are as follows:

BEAM TYPE	CP1-A	CP2-E
Built-up 80x80x15.68	04 (2 flanges, 2 web)	10 (5 flanges, 5 web)
W 130x130x28.1	04 (2 flanges, 2 web)	10 (5 flanges, 5 web)
Joists 102x102x23	04 (2 flanges, 2 web)	10 (5 flanges, 5 web)
UB 178x102x19	04 (2 flanges, 2 web)	10 (5 flanges, 5 web)
UC 152x152x30	04 (2 flanges, 2 web)	NA

REINFORCING REBARS

- (1) Reinforcing samples must be taken in the same batch with reinforcement used for concrete slab.
(2) All specimens shall be used one type of reinforcement.
(3) Numbers of RB for each reinforcement batch are 20.

DRAWING TITLE	
MATERIAL SAMPLES	
DRAWING NO : M-01	2011-11
DESIGNER: NGUYEN TUAN TRUNG	

Appendix C The Bailey-BRE method

(Bailey and Moore 2000; Bailey 2003; Bailey and Toh 2007)

This Appendix summarizes the Bailey-BRE method for reference only. More details can be found in Bailey and Moore (2000a; 2000b), Bailey (2003), Bailey and Toh (2007).

The calculation procedure can be summarised as follows:

- (1) Calculate the yield line load of the slab panel
- (2) Calculate the parameters k and b which define the in-plane stress distribution
- (3) Calculate the contribution of membrane forces to load bearing capacity for Elements 1 and 2.
- (4) Calculate the effect of membrane forces on bending resistance for Elements 1 and 2.
- (5) Calculate the overall enhancement factor.

The load bearing capacity of a two-way spanning simply supported slab, with no in-plane horizontal restraint at its edges, is greater than that calculated using the conventional yield-line theory. The load capacity is enhanced by tensile membrane action mobilised in the slab at large displacement and by the increase of the yield moment in the outer regions of the slab, where compressive stresses occur across the yield lines (see **Fig. C.2**).

Fig. C.1 Assumed failure modes for composite floor

It is assumed that at ultimate conditions the yield line pattern will be as shown in **Fig. C.1(a)** and that failure will occur due to fracture of the mesh across the short span at the centre of the slab. A second mode of failure might, in some cases, occur due to crushing of the concrete in the corners of the slab where high compressive in-plane forces occur as shown by **Fig. C.1(b)**. This mode of failure is discussed in the end of this Appendix.

Fig. C.2 Rectangular slab simply supported on four edges showing in-plane forces across the yield lines due to TMA

Fig. C.2 shows a rectangular simply supported slab and the lower bound yield-line pattern subjected to uniformly distributed loading. The intersection of the yield lines is defined by the parameter n which is given by:

$$n = \frac{1}{2\sqrt{\mu a^2}} \left[\sqrt{3\mu a^2 + 1} - 1 \right] \leq 0.5$$

n is limited to maximum of 0.5 resulting in a valid yield line pattern; a is the aspect ratio of the slab (L/l); μ is the ratio of the yield moment capacity of the slab in orthogonal directions ($\mu \leq 1.0$).

The yield line load of the slab based on the formation of these yield lines is given by:

$$p_y = \frac{24\mu M_0}{l^2} \left[\sqrt{3 + \frac{1}{(\sqrt{\mu}a)^2}} - \frac{1}{\sqrt{\mu}a} \right]^{-2}$$

Hayes (1968a) noted that assuming rigid-plastic behaviour, only rigid body translations and rotations are allowed. Further assumptions that the neutral axes along the yield lines are straight lines and that the concrete stress-block is rectangular. It means that the variations in membrane forces along the yield lines become linear (**Fig. C.3**). These assumptions and the resulting distribution of membrane forces were also adopted by Bailey.

Fig. C.3 In-plane stress distribution for the elements 1 and 2

Derivation of an expression for parameter k

Considering the equilibrium of the in-plane forces T_1 , T_2 and C acting on Element 1 allows the following relationships to be derived, where ϕ is the angle defining the yield line pattern.

$$\frac{T_1}{2} \sin \phi = C - T_2 \quad (\text{C.1})$$

$$x = \frac{L}{2} \cos \phi - \frac{L/2 - nL}{\cos \phi}; \quad y = \frac{1}{1+k} \sqrt{(nL)^2 + \frac{l^2}{4}};$$

Fig. C.4 In-plane stress distribution along yield line CD

Fig. C.4 shows the geometry of the stress distribution along yield line CD. From **Figs. C.3** and **C.4**, one can have:

$$\begin{aligned}
 T_1 &= bKT_0 (L - 2nL) \\
 T_2 &= \frac{bKT_0}{2} \left(\frac{1}{1+k} \right) \sqrt{(nL)^2 + \frac{l^2}{4}} \\
 C &= \frac{kbKT_0}{2} \left(\frac{k}{1+k} \right) \sqrt{(nL)^2 + \frac{l^2}{4}} \\
 \sin \varphi &= \frac{nL}{\sqrt{(nL)^2 + \frac{l^2}{4}}}
 \end{aligned}$$

Substituting the above expressions into Eq. (C.1) gives:

$$\frac{bKT_0 (L - 2nL)}{2} \cdot \frac{nL}{\sqrt{(nL)^2 + \frac{l^2}{4}}} = \frac{kbKT_0}{2} \left(\frac{k}{1+k} \right) \sqrt{(nL)^2 + \frac{l^2}{4}} - \frac{bKT_0}{2} \left(\frac{1}{1+k} \right) \sqrt{(nL)^2 + \frac{l^2}{4}}$$

This gives:

$$k = \frac{4na^2(1-2n)}{4n^2a^2 + 1} + 1 \quad (C.2)$$

Derivation of an expression for parameter b

Considering the fracture of the reinforcement across the short span of the slab, an expression for the parameter b can be obtained. The line EF shown in **Fig. C.5** represents the location of the mesh fracture, which will result in a full depth crack across the slab. An upper bound solution for the in-plane moment of resistance along the line EF can be obtained by assuming that all the reinforcement along the section

$$f_u = 1.1 f_y$$
$$T_2 \left[\left(\frac{L}{2} \cos \phi - \frac{\left(\frac{L}{2} - nL \right)}{\cos \phi} \right) \frac{1}{\tan \phi} - \frac{1}{3} \left(\frac{1}{k+1} \right) \sqrt{(nL)^2 + \frac{l^2}{4}} \right] \\ + C \left[\frac{L}{2} \sin \phi - \frac{1}{3} \left(\frac{k}{1+k} \right) \sqrt{(nL)^2 + \frac{l^2}{4}} \right] \\ + S \frac{L}{2} \cos \phi - \frac{T_1}{2} \left[\frac{1}{2} \left(\frac{L}{2} - nL \right) \right] = \frac{1.1 T_0 l^2}{8}$$
$$Ab + Bb + Cb - Db = \frac{1.1l^2}{8K}$$
$$b = \frac{1.1l^2}{8K(A+B+C-D)} \quad (\text{C.3})$$

$$A = \frac{1}{2} \left(\frac{1}{1+k} \right) \left[\frac{l^2}{8n} - \frac{(L/2 - nL)}{nL} \left((nL)^2 + \frac{l^2}{4} \right) - \frac{1}{3} \left(\frac{1}{1+k} \right) \left((nL)^2 + \frac{l^2}{4} \right) \right]$$

$$B = \frac{1}{2} \left(\frac{k^2}{1+k} \right) \left[\frac{nL^2}{2} - \frac{k}{3(1+k)} \left((nL)^2 + \frac{l^2}{4} \right) \right]$$

$$C = \frac{l^2}{16n} (k-1)$$

$$D = \left(\frac{L}{2} - nL \right) \left(\frac{L}{4} - \frac{nL}{2} \right)$$

Membrane forces

The load bearing capacity for Elements 1 and 2 of the slab can be determined by considering the contribution of the membrane forces to the resistance and the increase in bending resistance across the yield lines separately. These effects are expressed in terms of an enhancement factor, to be applied to the lower bound yield line resistance. The effect of the inplane shear S (**Fig. C.3**) or any vertical shear on the yield line was initially ignored, resulting in two unequal loads being calculated for Elements 1 and 2 respectively. An averaged value was then calculated, considering contribution of the shear forces.

Contribution of membrane forces to load bearing capacity

a) Element 1

Fig. C.6 Calculating the moment caused by the membrane force

We have:

$$M_{1m} = KT_0 Lbw \left((1-2n) + \frac{n(3k+2)}{3(1+k)^2} - \frac{nk^3}{3(1+k)^2} \right)$$

where M_{1m} is the moment about the support due to membrane forces for element 1.

The above formulation defines the contribution from the membrane forces to the load bearing capacity that needs to be added to the contribution due to the enhanced bending capacity in the areas where the slab is experiencing compression forces. For simplicity, the contribution from the membrane forces and enhanced bending action is related to the normal yield line load. This allows an enhancement factor to be calculated for both the membrane force and the enhanced bending moments. These enhancement factors can finally be added to give the overall enhancement of the slab due to membrane action. Dividing M_{1m} by $\mu M_o L$, the resistance moment of the slab, when no axial force is present, allows the effect of TMA to be expressed as an enhancement of yield line resistance. The value of $\mu M_o L$ is obtained based on **Fig. C.7**.

Fig. C.7 Calculation of the moment resistance

The bending moments μM_o and M_o per unit width of slab in each orthogonal direction are given by:

$$\mu M_o = KT_o z_1 = KT_o d_1 \left(\frac{3 + (g_o)_1}{4} \right)$$

$$M_o = T_o z_2 = T_o d_2 \left(\frac{3 + (g_o)_2}{4} \right)$$

where $(g_o)_1$ and $(g_o)_2$ are parameters which define the flexural stress block in short and long spans, respectively.

The enhancement factor, e_{1m} , is given by:

$$e_{1m} = \frac{M_{1m}}{\mu M_o L} = \frac{4b}{3 + (g_o)_1} \left(\frac{w}{d_1} \right) \left((1 - 2n) + \frac{n(3k + 2)}{3(1 + k)^2} - \frac{nk^3}{3(1 + k)^2} \right) \quad (C.4)$$

b) *Element 2*

The moment about the support due to the membrane forces is given by:

$$M_{2m} = KT_0 lbw \left(\frac{2+3k}{6(1+k)^2} - \frac{k^3}{6(1+k)^2} \right)$$

The effect of tensile membrane action can be expressed as an enhancement of yield line resistance by dividing the moment about the support due to membrane action, M_{2m} by the moment resistance in the longitudinal direction, when no axial force is present, $M_o l$, which results in:

$$e_{2m} = \frac{M_{2m}}{M_o l} = \frac{4bK}{3+(g_o)_2} \left(\frac{w}{d_2} \right) \left(\frac{2+3k}{6(1+k)^2} - \frac{k^3}{6(1+k)^2} \right) \quad (C.5)$$

The effect of the membrane forces on the bending resistance along the yield lines is evaluated by considering the yield criterion when axial load is also present, as given by Wood (1961). In the case of the short span the bending moment in the presence of an axial force is given by:

$$\frac{M_N}{\mu M_o} = 1 + \alpha_1 \left(\frac{N}{KT_o} \right) - \beta_1 \left(\frac{N}{KT_o} \right)^2 \quad (C.6)$$

where:

$$\alpha_1 = \frac{2(g_o)_1}{3+(g_o)_1}; \beta_1 = \frac{1-(g_o)_1}{3+(g_o)_1}$$

Similarly for the long span,

$$\frac{M_N}{M_o} = 1 + \alpha_2 \left(\frac{N}{T_o} \right) - \beta_2 \left(\frac{N}{T_o} \right)^2 \quad (C.7)$$

where:

$$\alpha_2 = \frac{2(g_o)_2}{3+(g_o)_2}; \beta_2 = \frac{1-(g_o)_2}{3+(g_o)_2}$$

Effect of membrane forces on bending resistance

a) Element 1

The effect of the membrane forces on the bending resistance is considered separately for each yield line. Along the yield line BC:

$$\left(\frac{M_N}{M_0} \right)_{BC} = 1 - \alpha_1 b - \beta_1 b^2$$

Along the yield line AB, the membrane force across the yield line, at a distance of x from B is given by:

$$N_x = -bKT_0 + \frac{x}{nL}(k+1)bKT_0 = bKT_0 \left(\frac{x}{nL}(k+1) - 1 \right)$$

Substitution into Eq. (C.6) gives, for yield lines AB and CD:

$$2 \int_0^{nL} \frac{M}{M_0} dx = 2nL \left[1 + \frac{\alpha_1 b}{2}(k-1) - \frac{\beta_1 b^2}{3}(k^2 - k + 1) \right]$$

The enhancement of bending resistance due to membrane forces on Element 1 is given by:

$$e_{1b} = \frac{M}{\mu M_0 L} = 2n \left[1 + \frac{\alpha_1 b}{2}(k-1) - \frac{\beta_1 b^2}{3}(k^2 - k + 1) \right] + (1-2n)(1 - \alpha_1 b - \beta_1 b^2) \quad (C.8)$$

b) Element 2

Referring to **Fig. C.8** for element 2, the force at a distance y from B can be expressed as:

$$N_y = -bKT_0 + \frac{y}{l/2}(k+1)bKT_0 = bKT_0 \left(\frac{2y}{l}(k+1) - 1 \right)$$

Fig. C.8 Forces applied to element 2

Resulting in,

$$2 \int_0^{l/2} \frac{M}{M_0} dx = l \left[1 + \frac{\alpha_2 b}{2} (k-1) - \frac{\beta_2 b^2}{3} (k^2 - k + 1) \right]$$

which gives the enhancement factor due to the effect of the membrane forces on the bending resistance according to the following formulation:

$$e_{2b} = \frac{M}{M_0 l} = 1 + \frac{\alpha_2 b K}{2} (k-1) - \frac{\beta_2 b^2 K}{3} (k^2 - k + 1) \quad (C.9)$$

Combined enhancement factor for each element

Eqs. (C.4), (C.5), (C.8) and (C.9) provide the contribution to the load bearing capacity due to the membrane forces and the effect of the membrane forces on the bending resistance of the slab. Consequently, the combined enhancement factor is obtained for each element as follows:

$$e_1 = e_{1m} + e_{1b}$$

$$e_2 = e_{2m} + e_{2b}$$

As stated earlier, the values e_1 and e_2 calculated based on the equilibrium of elements 1 and 2 will not be the same and Hayes (1968a) suggests that these differences can be explained by the effect of the vertical or in-plane shear and that the overall enhancement is given by:

$$e = e_1 - \frac{e_1 - e_2}{1 + 2\mu a^2}$$

Compressive failure of concrete

The abovementioned enhancement factor was derived by considering tensile failure of the mesh reinforcement. However, compressive failure of the concrete in the proximity of the slab corners must also be considered as a possible mode of failure, which in some cases may precede mesh fracture. This was achieved by limiting the value of the parameter ‘ b ’, which represents the magnitude of the in-plane stresses.

According to **Fig. C.3**, the maximum in-plane compressive force at the corners of the slab is given by $kbKT_0$. The compressive force due to bending should also be considered. By assuming that the maximum stress-block depth is limited to $0.45d$, and adopting an average effective depth to the reinforcement in both orthogonal directions results in:

$$kbKT_0 + \left(\frac{KT_0 + T_0}{2} \right) = 0.85f_{ck} \times 0.45 \left(\frac{d_1 + d_2}{2} \right)$$

where f_{ck} is the concrete cylinder strength.

Solving for the constant b gives:

$$b = \frac{1}{kKT_0} \left[0.85f_{ck} 0.45 \left(\frac{d_1 + d_2}{2} \right) - T_0 \left(\frac{K+1}{2} \right) \right] \quad (\text{C.10})$$

The constant b is then taken as the minimum value given by the Eqs. (C.3) and (C.10).

Reference

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Appendix D Derivation of the Proposed Semi-Analytical Model

This Appendix presents the derivations of the semi-analytical model proposed by the author.

The only unknown in the following equations is the load-bearing capacity of the slab, q . Parts of this load apply to elements 1 and 2 are q_1 and q_2 , respectively. Eq. (7.26) aims to find q_1 , while Eq. (7.30) to find q_2 . Based on those values, the overall enhancement factor can be calculated.

Eq. (7.26)

$$\begin{aligned}
 & 2 \int_0^{L/2-nL} bKT_0 (w_m - w_{zx}^{(1)}) dx \\
 & + 2 \int_{L/2-nL}^{L/2} \left[bKT_0 - \frac{x - (L/2 - nL)}{nL} (k+1) bKT_0 \right] \times \left[\frac{L/2 - x}{nL} w_m - w_{zx}^{(2)} \right] dx \\
 & - q(L - 2nL) \frac{l}{2} \frac{l}{4} - q \times nL \times \frac{l}{2} \times \frac{l}{6} = 0 \\
 & \leftrightarrow \bar{A} + \bar{B} - \left(\frac{1}{8} - \frac{n}{6} \right) qLl^2 = 0
 \end{aligned}$$

Set:

$$\bar{A} = 2bKT_0 \int_0^{L/2-nL} \left[w_m - \left[w_{zx\max} - \frac{1}{EI_1} \left[M_{xo} \times \frac{x^2}{2} - q_1 \frac{x^4}{24} \right] + \alpha T_{,zb1} \left(\frac{L^2}{8} - \frac{x^2}{2} \right) \right] \right] dx$$

Based on mechanics, we know that:

$$w_{zx\max} = \frac{q_1}{EI_1} \left[\frac{1}{24} \left[8(nL)^2 + 12(nL)(L - 2nL) + 3(L - 2nL)^2 \right] \times \frac{1}{8} L^2 - \frac{1}{24} \left(\frac{L}{2} \right)^4 + \frac{(nL)^4}{120} \right]$$

$$\text{Set: } w_{zx\max} = \frac{q_1}{EI_1} \alpha_1$$

$$M_{xo} = \frac{q_1}{24} \left[8(nL)^2 + 12(nL)(L - 2nL) + 3(L - 2nL)^2 \right] = \beta_1 q_1$$

We have:

$$\bar{A} = 2bKT_0 \int_0^{L/2-nL} \left[w_m - \left[\alpha_1 \frac{q_1}{EI_1} - \frac{1}{EI} \left(\beta_1 q_1 \frac{x^2}{2} - q_1 \frac{x^4}{24} \right) + \alpha T_{,zb1} \left(\frac{L^2}{8} - \frac{x^2}{2} \right) \right] \right] dx$$

$$\boxed{\bar{A} = 2bKT_0 w_m \left(\frac{L}{2} - nL \right) - 2bKT_0 \frac{q_1}{EI_1} \times \gamma_1 - 2bKT_0 \times \delta_1}$$

with

$$\alpha_1 = \frac{1}{24} \left[8(nL)^2 + 12(nL)(L - 2nL) + 3(L - 2nL)^2 \right] \times \frac{1}{8} L^2 - \frac{1}{24} \left(\frac{L}{2} \right)^4 + \frac{(nL)^4}{120}$$

$$\beta_1 = \frac{1}{24} \left[8(nL)^2 + 12(nL)(L - 2nL) + 3(L - 2nL)^2 \right]$$

$$\gamma_1 = \alpha_1 \left(\frac{L}{2} - nL \right) - \frac{\beta_1}{6} \left(\frac{L}{2} - nL \right)^3 + \frac{1}{120} \left(\frac{L}{2} - nL \right)^5$$

$$\delta_1 = \alpha T_{,zb1} \left[\frac{1}{8} L^2 \left(\frac{L}{2} - nL \right) - \frac{1}{6} \left(\frac{L}{2} - nL \right)^3 \right]$$

$$\begin{aligned} \bar{B} &= 2 \int_{L/2-nL}^{L/2} \left[bKT_0 - \frac{x - (L/2 - nL)}{nL} (k+1) bKT_0 \right] \times \left[\frac{L/2 - x}{nL} w_m - w_{zx}^{(2)} \right] dx \\ &= 2bKT_0 \int_{L/2-nL}^{L/2} \left[\frac{L/2 - x}{nL} w_m - w_{zx}^{(2)} - \frac{x - (L/2 - nL)}{nL} \frac{L/2 - x}{nL} (k+1) w_m + \frac{x - (L/2 - nL)}{nL} (k+1) w_{zx}^{(2)} \right] dx \\ &= 2bKT_0 (\bar{C} + \bar{D} + \bar{E} + \bar{F}) \end{aligned}$$

where

$$\begin{aligned} \bar{C} &= \int_{L/2-nL}^{L/2} \frac{L/2 - x}{nL} w_m dx; \quad \bar{D} = \int_{L/2-nL}^{L/2} (-w_{zx}^{(2)}) dx; \\ \bar{E} &= \int_{L/2-nL}^{L/2} -\frac{x - (L/2 - nL)}{nL} \frac{L/2 - x}{nL} (k+1) w_m dx; \quad \bar{F} = \int_{L/2-nL}^{L/2} \frac{x - (L/2 - nL)}{nL} (k+1) w_{zx}^{(2)} dx \end{aligned}$$

$$\boxed{\bar{C} = w_m L \left[-\frac{1}{8n} + \frac{1}{2} + \frac{1}{2n} \left(\frac{1}{2} - n \right)^2 \right]}$$

$$\boxed{\bar{D} = \frac{q_1}{EI_1} \left(-\alpha_2 + \frac{(nL)^5}{720} \right) - \alpha T_{,zb1} \times \left[\frac{1}{8} L^2 (nL) - \frac{1}{6} \left(\left(\frac{L}{2} \right)^3 - \left(\frac{L}{2} - nL \right)^3 \right) \right]}$$

with

$$\alpha_2 = \left(\alpha_1 \frac{L}{2} - \frac{\beta_1}{6} \frac{L^3}{8} + \frac{1}{120} \frac{L^5}{32} \right) - \gamma_1$$

$$\boxed{\bar{E} = -\frac{(k+1)}{(nL)^2} w_m \times L^3 \times \beta_2}$$

$$\boxed{\bar{F} = \alpha_3 \frac{q_1}{EI_1} + \frac{k+1}{nL} \alpha T_{,zb1} \left(\frac{1}{16} L^2 (nL)^2 - \frac{1}{8} (nL)^4 \right) + \frac{k+1}{nL} \left(\frac{L}{2} - nL \right) \bar{D}}$$

with

$$\alpha_3 = \frac{k+1}{nL} \left[\gamma_2 - \frac{1}{120} \left(\frac{1}{7} (nL)^6 + \frac{1}{6} (nL)^5 \left(\frac{L}{2} - nL \right) \right) \right]$$

$$\gamma_2 = \left(\frac{\alpha_1}{2} \frac{L^2}{4} - \frac{\beta_1}{8} \frac{L^4}{16} + \frac{1}{144} \frac{L^6}{64} \right) - \left(\frac{\alpha_1}{2} \left(\frac{L}{2} - nL \right)^2 - \frac{\beta_1}{8} \left(\frac{L}{2} - nL \right)^4 + \frac{1}{144} \left(\frac{L}{2} - nL \right)^6 \right)$$

Replacing A , B and regrouping, Eq. (7.26) becomes:

$$\bar{A} + \bar{B} - \left(\frac{1}{8} - \frac{n}{6} \right) q L l^2 = 0$$

$$2bKT_0 w_m \left(\frac{L}{2} - nL \right) - 2bKT_0 \frac{q_1}{EI_1} \times \gamma_1 - 2bKT_0 \times \delta_1$$

$$+ 2bKT_0 (\bar{D} + \bar{F}) + 2bKT_0 (\bar{C} + \bar{E}) - \left(\frac{1}{8} - \frac{n}{6} \right) L l^2 q = 0$$

$$\text{Set } \boxed{\beta_3 = 1 + \frac{k+1}{nL} \left(\frac{L}{2} - nL \right)}$$

Replace $q_1 = ql/2$, Eq. (7.26) becomes Eq. (7.27):

$$\boxed{2bKT_0 \times \left[w_m \left(\frac{L}{2} - nL \right) - \delta_1 + (\bar{C} + \bar{E}) - \beta_3 \times \alpha T_{,zb1} \times \left[\frac{1}{8} L^2 (nL) - \frac{1}{6} \left(\left(\frac{L}{2} \right)^3 - \left(\frac{L}{2} - nL \right)^3 \right) \right] \right.}$$

$$\left. + \frac{k+1}{nL} \alpha T_{,zb1} \left(\frac{1}{16} L^2 (nL)^2 - \frac{1}{8} (nL)^4 \right) \right]$$

$$= ql \left[\left(\frac{1}{8} - \frac{n}{6} \right) Ll + bKT_0 \frac{1}{EI_1} (\gamma_1 - \alpha_3) - bKT_0 \times \beta_3 \frac{1}{EI_1} \left(-\alpha_2 + \frac{(nL)^5}{720} \right) \right]$$

Where

$$\alpha_1 = \frac{1}{24} \left[8(nL)^2 + 12(nL)(L - 2nL) + 3(L - 2nL)^2 \right] \times \frac{1}{8} L^2 - \frac{1}{24} \left(\frac{L}{2} \right)^4 + \frac{(nL)^4}{120}$$

$$\beta_1 = \frac{1}{24} \left[8(nL)^2 + 12(nL)(L - 2nL) + 3(L - 2nL)^2 \right]$$

$$\gamma_1 = \alpha_1 \left(\frac{L}{2} - nL \right) - \frac{\beta_1}{6} \left(\frac{L}{2} - nL \right)^3 + \frac{1}{120} \left(\frac{L}{2} - nL \right)^5$$

$$\delta_1 = \alpha T_{,zbl} \left[\frac{1}{8} L^2 \left(\frac{L}{2} - nL \right) - \frac{1}{6} \left(\frac{L}{2} - nL \right)^3 \right]$$

$$\alpha_2 = \left(\alpha_1 \frac{L}{2} - \frac{\beta_1}{6} \frac{L^3}{8} + \frac{1}{120} \frac{L^5}{32} \right) - \gamma_1$$

$$\bar{C} = w_m L \left[-\frac{1}{8n} + \frac{1}{2} + \frac{1}{2n} \left(\frac{1}{2} - n \right)^2 \right]$$

$$\bar{E} = -\frac{(k+1)}{(nL)^2} w_m \times L^3 \times \beta_2$$

$$\beta_2 = \frac{1}{48} - \frac{1}{8} \left(\frac{1}{2} - n \right) + \frac{1}{4} \left(\frac{1}{2} - n \right)^2 - \frac{1}{6} \left(\frac{1}{2} - n \right)^3$$

$$\alpha_3 = \frac{k+1}{nL} \left[\gamma_2 - \frac{1}{120} \left(\frac{1}{7} (nL)^6 + \frac{1}{6} (nL)^5 \left(\frac{L}{2} - nL \right) \right) \right]$$

$$\gamma_2 = \left(\frac{\alpha_1}{2} \frac{L^2}{4} - \frac{\beta_1}{8} \frac{L^4}{16} + \frac{1}{144} \frac{L^6}{64} \right) - \left(\frac{\alpha_1}{2} \left(\frac{L}{2} - nL \right)^2 - \frac{\beta_1}{8} \left(\frac{L}{2} - nL \right)^4 + \frac{1}{144} \left(\frac{L}{2} - nL \right)^6 \right)$$

$$\beta_3 = 1 + \frac{k+1}{nL} \left(\frac{L}{2} - nL \right)$$

Eq. (7.30)

$$2 \int_0^{l/2} \left[bKT_0 - \frac{y}{l/2} (k+1) bKT_0 \right] \times \left[\left(1 - \frac{y}{l/2} \right) w_m - w_{zy} \right] dy - \frac{1}{6} ql \times (nL)^2 = 0$$

Set: $A^* = 2 \int_0^{l/2} \left[bKT_0 - \frac{y}{l/2} (k+1) bKT_0 \right] \times \left[\left(1 - \frac{y}{l/2} \right) w_m - w_{zy} \right] dy$

Based on mechanics, we know that:

$$0 \leq y \leq l/2:$$

$$w_{zy}(y) = w_{zy\max} - \frac{1}{EI_2} \left[M_{yB} \times \frac{y^2}{2} - q_2 \frac{y^4}{24} + \frac{2}{l} q_2 \times \frac{y^5}{120} \right] + \alpha T_{,zb2} \left(\frac{l^2}{8} - \frac{y^2}{2} \right)$$

$$w_{vy\max} = \frac{1}{120} \frac{q_2 l^4}{EI_2} \quad ; \quad M_{yA} = \frac{1}{12} q_2 l^2$$

$$\begin{aligned} A^* &= 2 \int_0^{l/2} \left[bKT_0 - \frac{y}{l/2} (k+1) bKT_0 \right] \times \left[\left(1 - \frac{y}{l/2} \right) w_m - w_{zy} \right] dy \\ &= 2bKT_0 \int_0^{l/2} \left[1 - \frac{2y}{l} (k+1) \right] \times \left[\left(1 - \frac{2y}{l} \right) w_m - w_{zy} \right] dy \\ &= 2bKT_0 \left[\int_0^{l/2} \left(1 - \frac{2y}{l} \right) w_m dy - \int_0^{l/2} w_{zy} dy - \int_0^{l/2} \frac{2y}{l} (k+1) \left(1 - \frac{2y}{l} \right) w_m dy + \int_0^{l/2} \frac{2y}{l} (k+1) w_{zy} dy \right] \end{aligned}$$

$$A_1 = \int_0^{l/2} \left(1 - \frac{2y}{l} \right) w_m dy = \frac{w_m l}{4}$$

$$B_1 = - \int_0^{l/2} w_{zy} dy = - \frac{61}{23040} \frac{q_2 l^5}{EI_2} - \alpha T_{,zb2} \frac{1}{24} l^3$$

$$C_1 = - (k+1) \frac{2}{l} w_m \times \frac{1}{24} l^2 = - \frac{1}{12} (k+1) w_m l$$

$$D_1 = \int_0^{l/2} \frac{2y}{l} (k+1) w_{zy} dy = \frac{31}{32256} (k+1) \frac{q_2}{EI_2} + (k+1) \alpha T_{,zb2} \frac{l^3}{64}$$

Thus Eq. (7.30) becomes:

$$\begin{aligned} &\frac{1}{6} ql \times (nL)^2 - 2bKT_0 \left[- \frac{61}{23040} \frac{q_2 l^5}{EI_2} + \frac{31}{32256} (k+1) \frac{q_2}{EI_2} \right] \\ &= 2bKT_0 \left[\frac{w_m l}{4} - \alpha T_{,zb2} \frac{1}{24} l^3 - \frac{1}{12} (k+1) w_m l + (k+1) \alpha T_{,zb2} \frac{l^3}{64} \right] \end{aligned}$$

Substitute: $q_2 = q(nL)$

$$\begin{aligned} &\left[q \left[\frac{1}{6} (nL)^2 l - 2bKT_0 \frac{1}{EI_2} (nL) \left[- \frac{61}{23040} l^5 + \frac{31}{32256} (k+1) \right] \right] \right] \\ &= 2bKT_0 \left[\frac{w_m l}{4} - \alpha T_{,zb2} \frac{1}{24} l^3 - \frac{1}{12} (k+1) w_m l + (k+1) \alpha T_{,zb2} \frac{l^3}{64} \right] \end{aligned} \quad (7.31)$$

Appendix E Calculation of the load-bearing capacity enhanced by TMA for S1

Defining the ultimate load of a two-way slab supported by four flexible beams

1/ Data

$L = 2250$	mm	$l = 2250$	mm
$h = 55$	mm	$width = 1000$	mm
$d_x = 38$	mm	$d_y = 35$	mm

Bottom reinf. in x dir. (long span)	#3 @80	$A_{sx} = 84.82$	mm ²	0.22%
Bottom reinf. in y dir. (short span)	#3 @80	$A_{sy} = 84.82$	mm ²	0.24%

Concrete

$ST =$ 172	$^{\circ}\text{C}$	$SB =$ 630	$^{\circ}\text{C}$		
$E_{cm} =$ 32389	MPa	$f_{cm} =$ 36	N/mm ²	$\alpha =$ 1.20E-05	mm/mm K
$E_{cT} =$ 27487	MPa	$f_{cmT} =$ 35	N/mm ²	$I_c =$ 1.39E+07	mm ⁴

Reinforcement properties

$\theta_m = 391$	^0C		
$f_{yk} = 543$	N/mm^2	$f_u = 771$	N/mm^2
$f_{ykT} = 543$	N/mm^2	$f_{uT} = 771$	N/mm^2

Secondary beam

lary beam		$\alpha = 1.30E-05$ mm/mm K			
		$PT = 172$	0C	$PB = 752$	0C
$h_l = 80$	mm	$b_{fl} = 80$	mm	$t_{fl} = 9.14$	mm
$I_l = 2.15E+06$	mm ⁴	$W_{pll} =$	mm ³	$A = 2020$	mm ²
$E_s = 206900$	MPa	$f_{yk} = 435$	MPa		
$E_{sT} = 192003.2$	MPa	$f_{ykT} = 435$	MPa	$T_{,zbl} = 4.30$	$^0C/mm$

Main beam

beam		$\alpha = 1.30E-05 \text{ mm/mm K}$			
		$MT = 172$	$^{\circ}\text{C}$	$MB = 756$	$^{\circ}\text{C}$
$h_2 = 131$	mm	$b_{f2} = 128$	mm	$t_{f2} = 10.77$	mm
$I_2 = 1.10E+07$	mm ⁴	$W_{pl2} = 191520$	mm ³	$A = 3520$	mm ²
$E_s = 197500$	MPa	$f_{yk} = 302$	MPa		
$E_{sT} = 183280$	MPa	$f_{ykT} = 302$	MPa	$T_{,zb2} = 3.14$	$^{\circ}\text{C/mm}$

Deflection at failure

$w_{failure} = 131$	mm
---------------------	----

2/ Yield load

$$\begin{array}{llll}
 M_0 = 1.72 & \text{kNm} & \mu M_0 = 1.58 & \text{kNm} \\
 M_{0T} = 1.71 & \text{kNm} & \mu M_{0T} = 1.58 & \text{kNm} \\
 T_{0T} = 46.1 & \text{kN} & KT_{0T} = 46.1 & \text{kN} \\
 \mu = 0.92 & & K = 1.00 & a = 1.0 \\
 n = 0.49 & & nL/1000 = 1.10 & \text{sqrt}((nL)^2 + l^2/4) = 1.57 \\
 & & (L-2nL) = 0.05 &
 \end{array}$$

Parameters determined inplane stress distribution

$$\begin{array}{ll}
 k = 1.02 & \\
 A = 0.21 & B = 0.21 \\
 C = 0.01 & D = 0.00 \\
 b = 1.62 &
 \end{array}$$

$$\text{Yield line load} \quad p_{y,20} = 7.81 \text{ kN/m}^2 \quad p_{y,\theta} = 7.80 \text{ kN/m}^2$$

3/ Contribution of membrane forces to load bearing capacity

Element 1

Parameters

$$\begin{array}{lll}
 \alpha_1 = 0.22 & \beta_1 = 0.43 & \gamma_1 = 0.01 \\
 \alpha_2 = 0.15 & \beta_2 = 0.020 & \gamma_2 = 0.07 \\
 \alpha_3 = 0.12 & \beta_3 = 1.04 & \delta_1 = 0.00 \\
 \bar{C} = 0.07 & \bar{E} = -0.05 &
 \end{array}$$

Therefore

$$\begin{array}{lll}
 0.513 & x q_{1m}^* = 2.90 & \text{kN/m}^2 \\
 & q_{1m}^* = 5.66 & \text{kN/m}^2 \\
 & e_{1m}^* = 0.726 &
 \end{array}$$

Element 2

$$\begin{array}{lll}
 0.467 & x q_{2m}^* = 2.88 & \\
 & q_{2m}^* = 6.17 & \text{kN/m}^2 \\
 & e_{2m}^* = 0.791 &
 \end{array}$$

4/ Effect of membrane forces on bending resistance

$$\text{Element 1} \quad (g_0)_1 = 0.912 \quad e_{1b} = 0.971$$

$$\text{Element 2} \quad (g_0)_2 = 0.919 \quad e_{2b} = 0.990$$

5/ Load bearing capacity of the slab

$$\begin{array}{l}
 e_1^* = 1.70 \\
 e_2^* = 1.78
 \end{array}$$

$$\begin{array}{llll}
 \text{Thus} & e = 1.73 & p = 13.46 & \text{kN/m}^2 \\
 & \text{Bailey's prediction} & p = 15.82 & \text{kN/m}^2 \quad 14.9\%
 \end{array}$$

6/ Required bending moment of edge beam at mid-span

$$\text{Number of interior beams} \quad n = 0$$

$$L = 2250 \quad \text{mm}$$

$$l = 2250 \quad \text{mm}$$

Case 1: Yield line parallel to the unprotected beam

Test load $p = 15.6 \text{ kN/m}^2$

Effective width of main beam (EC4)

Internal side

$$b_{eff1} = \min(l_e/8, b_i) = 281.3 \quad \text{mm}$$

End support side

$$b_{e2} = 281.3$$

$$b_{eff2} = \beta_2 b_{e2} = 210.9$$

$$b_{eff} = b_{eff1} + b_{eff2} + b_0 = 492.2 \quad \text{mm}$$

Effective length of yield line $L_{eff} = 1687.5 \quad \text{mm}$

Resistance of slab per unit length $M_0 = 1.72 \quad \text{kNm}$

Thus $M_{b,1} = 9.66 \quad \text{kNm}$

Bending resistance of MB in fire $M_{RD,1} = 19.83 \quad \text{kNm} > 9.66$

OK

Case 2: Yield line perpendicular to the unprotected beam

Effective width of PSB (EC4)

Internal side

$$b_{eff1} = \min(L_e/8, b_i) = 281.3 \quad \text{mm}$$

End support side

$$b_{e2} = 281.3$$

$$b_{eff2} = \beta_2 b_{e2} = 210.9$$

$$b_{eff} = b_{eff1} + b_{eff2} + b_0 = 492.2 \quad \text{mm}$$

Effective width of USB (EC4)

$$b_{eff} = 2 * \min(L_e / 8, b_i) + b_0 = 0 \quad \text{mm}$$

Effective length of yield line $l_{eff} = 1265.6 \quad \text{mm}$

Resistance of slab per unit length $\mu M_0 = 1.58 \quad \text{kNm}$

$$M_{hot} = 0.00 \quad \text{kNm}$$

(calculating as steel beams gives conservative results)

Thus $M_{b,2} = 10.11 \quad \text{kNm}$

Bending resistance of PSB in fire $M_{RD,1} = 16.31 \quad \text{kNm} \quad > \quad 10.11 \quad \text{OK}$