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LIMIT CAPACITY OF EDGE AND CORNER COMPOSITE SLABS IN FIRE CONDITIONS

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ABSTRACT

The usage of tensile membrane action for the design of composite floors in steel framed buildings in fire conditions has led to a number of simplified design solutions, like the Bailey-BRE method and the Slab Panel Method. These methods predict an enhancement of the slab's yield-line load capacity by membrane stresses, on the assumption of ideal vertical support at all edges. However the vertical support is provided by protected edge beams, which deflect under a combination of heat and loading. The loss of vertical edge support results in an eventual collapse of the entire structure. In recent years four collapse mechanisms of composite slab panels have been proposed. The current research improves upon the predicted failure time for edge slab panels and develops a failure mechanism for slab panels located at the corner of a building. The results are checked against finite element simulations.

Introduction

Research has shown that traditional methods for the design of steel-framed buildings in fire conditions are excessive. Both accidental fires and tests on full-scale buildings have shown that the practice of protecting all exposed steelwork for fire resistance is not necessary to ensure structural stability of a steel-framed building. By taking advantage of tensile membrane action (TMA) structural stability can be ensured with significant reductions in costs (Newman et al., 2006). TMA is a high-capacity load-bearing mechanism of thin composite floors under large deflections. The deflection generates radial tension in the centre of the slab balanced by a peripheral ring of compression. Due to its self-sustaining nature, horizontal edge restraint is not required to activate this load-bearing capacity, so that only two-way bending of the slab and vertical support at all edges are necessary. To incorporate this mechanism in design, a building floor is divided into several rectangular zones of low aspect ratio, called slab panels. These slab panels comprise unprotected composite beams in the interior and four edges which primarily resist vertical deflection (Bailey, 2001). The vertical support is usually provided by protected composite beams, which would normally be located on column gridlines, as shown Fig. 1.

In fire the unprotected intermediate beams heat up and lose their strength rapidly so that their loads are borne by the slab which undergoes two-way bending and large deflections. With increasing temperature, the deflection of the slab and its resistance increases until the tensile strength of the reinforcement is reached. The advantage of using tensile membrane action is that a large number of floor beams can be left unprotected, which significantly reduces building costs while providing quantifiable structural stability in fire conditions. TMA and whole-structural behaviour of buildings can be modelled in a three-dimensional framework with sophisticated finite element software such as *Vulcan* (Huang et al., 2003), ABAQUS (2010) and SAFIR (2011) which incorporate geometrical and material non-linear behaviour. Due to the unsuitability of finite element simulations for day-to-day analyses, as they can be time-consuming costly processes, simpler performance-based methods are often preferred for routine design.

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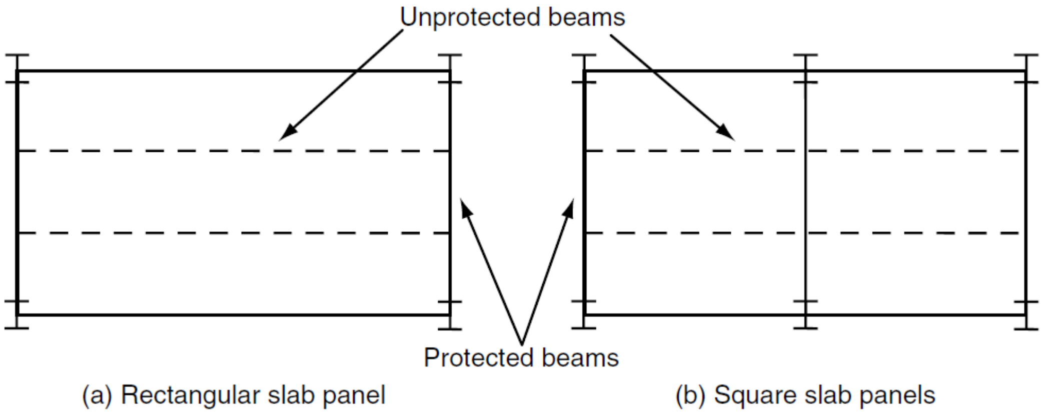

Figure 1. Rectangular and square slab panels (Abu et al., 2011)

Prominent amongst existing simplified methods incorporating tensile membrane action is the Bailey-BRE method (Bailey, 2001) which predicts composite slab capacity by calculating a membrane enhancement to the traditional yield-line load of the slab, by the assumption of continuous vertical support at all edges. However this support is provided by protected edge beams, which heat up and lose strength during fires. The loss of vertical support at the edges results in loss of tensile membrane action in the slab and an eventual failure of the whole slab panel. To encourage the use of simplified design approaches, four different collapse mechanisms have been proposed for composite slab panels in fire conditions (Abu et al., 2011). These mechanisms describe different potential collapse scenarios and provide equations to calculate the time of structural failure in fire.

Collapse Mechanisms

The structural failure of a slab panel can be calculated by work-balance equations under consideration of plastic folding, which allows collapse without generating membrane forces in the slab. Following the principle of virtual work the constant external work done by the applied fire limit state load is equated to the internal work done (by the components of the slab panel) which decreases over time with increasing temperature. The time step at which the external work exceeds the internal work defines the failure time of the slab panel for the postulated failure mode. The choice of which collapse mechanism occurs in fire depends on the aspect ratio of the slab, relative beam sizes, location of the slab panel within the building and the extent of the fire. Fig. 2 presents the four collapse mechanisms that have been proposed.

As the Bailey-BRE design approach is based on an isolated slab panel, Collapse Mechanism 1 examines the failure of this type of slab panel. Collapse Mechanism 2 addresses large compartments, such as open-plan offices where a large number of slab panels could be involved in the fire. It follows the general principles of Collapse Mechanism 1, but considers reinforcement continuity across two opposite edges.

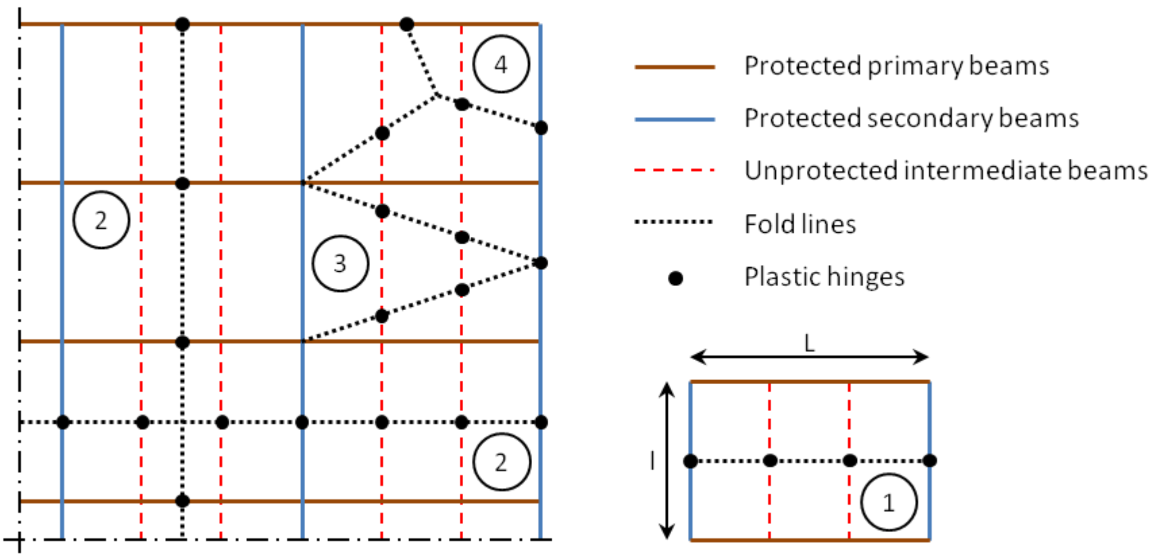

Figure 2. Proposed collapse mechanisms

Both mechanisms have been checked against finite element simulations and are found to be quite accurate (Abu et al., 2011). Edge and corner panels have to be treated differently, due to their different boundary conditions. Collapse Mechanism 3, for slab panels located at the edge of a building, has also been developed. However, the prediction was found to be grossly conservative in comparison with finite element simulations. The current paper improves upon this prediction, and also presents a new mechanism (Collapse Mechanism 4) for slab panels located at the corner of a building.

Analyses

All collapse mechanisms are verified with Vulcan (Huang et al., 2003) and checked against the Bailey-BRE limit and the conventional span/20 deflection limit for fire limit state. For comparison with the previous research, the design data for the previous example cases have been maintained. They are based on typical European office type loading: dead load (including concrete slab, reinforcement and beam self-weights) = 4.33 kN/m²; live load (maximum office load) = 5.0 kN/m²; trapezoidal decking profile with a trough depth of 60 mm; overall slab thickness of 130 mm and concrete cube strength of 40 N/mm², and consistent with the previous research (Abu et al., 2011). The composite floor beams were designed in accordance with EN1994-1-1 (CEN, 2004) and EN1994-1-2 (CEN, 2005), and the edge beams were protected to reach a maximum temperature of 550°C at 60 min of exposure to the Standard Fire. It should be noted that modern steel-framed design for fire conditions will specify much higher limiting temperatures (of the order of about 630°C).

Two slab panel sizes (12 m x 9 m and 12 m x 12 m, with properties listed in Table 1) were used for the verifications. In both slab panels intermediate beams were spaced at 3 m. The 12 m x 9 m slab panel and its beam sections were chosen for Collapse Mechanism 3, while the 12 m x 12 m slab panel was used to evaluate Collapse Mechanism 4.

Table 1. Slab panel design data

Slab Panel Size	Beam Type	Beam Section	Load Ratio	Temperature at 60 min	Span (m)
12 m x 9 m	Primary	610 x 305 x 179 UB	0.380	550°C	9
	Left Secondary	533 x 210 x 101 UB	0.413	548°C	12
	Right Secondary	406 x 178 x 67 UB	0.419	548°C	12
	Intermediate	457 x 152 x 67 UB	0.469	941°C	12
12 m x 12 m	Top Primary	610 x 305 x 149 UB	0.422	550°C	12
	Bottom Primary	838 x 292 x 194 UB	0.463	550°C	12
	Left Secondary	533 x 210 x 101 UB	0.446	548°C	12
	Right Secondary	475 x 191 x 67 UB	0.437	547°C	12
	Intermediate	406 x 178 x 74 UB	0.454	940°C	12

Results and Discussion

Collapse Mechanism 3

The predicted failure time of the previous research for Collapse Mechanism 3 was conservative and about 30% shy of the equivalent prediction of the Vulcan simulation. Likewise the related deflection-time plot of the simulation did not show a clear sign of failure. To enhance the prediction, the present study reviewed the proposed equations for slab panel failure and the *Vulcan* simulations. An artificial horizontal restraint was found in the simulations. As such, the simulation predicted a much higher capacity than would actually be available if the edges could move inwards. The moment capacity of the concrete slab at elevated temperatures was overestimated in the analytical prediction. The equation that considered the collapse of the panel involving the failure of the primary edge beam did not include the capacity of the unprotected intermediate beams. Both assumptions have now been incorporated in the new predictions. The new prediction of the analytical model now occurs 22 minutes earlier than the results of the new Vulcan simulations. Significant improvement of the failure prediction was obtained with the selection of an appropriate centre of compression for hogging moments along the continuous edges, as represented in Fig. 3.

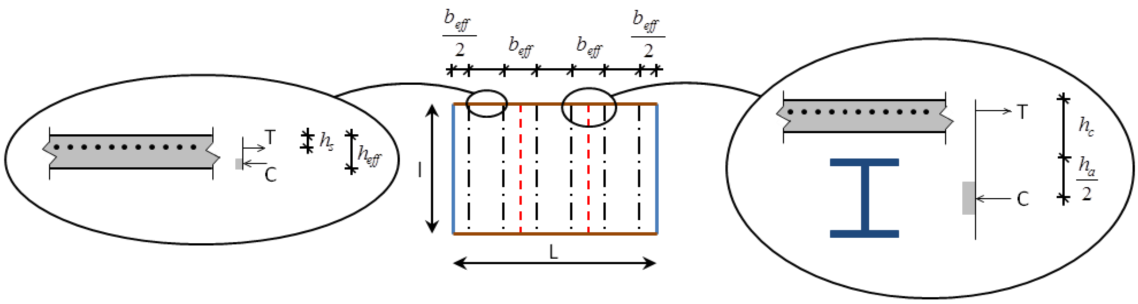

Figure 3. Different moment capacities

For the moment capacity along the edges of continuity, the total length of the edge was divided into areas for the effective width of each composite beam and areas that only contributed to the slab bending capacity. The areas that only contributed to the slab capacity were treated as having compression in the bottom regions of the concrete slab, while those areas that were part of the effective with of the composite beams had their centres of compression at the middle of the steel beams, on the assumption of simple connections. The larger leaver arm led to significant increase of moment capacity. These improvements were incorporated into the equations and checked against the new simulations, as shown in Fig. 4. The figure shows a 24-minute improvement in the failure prediction, as compared to the previous research. These improvements have also been incorporated in Collapse Mechanism 2, to improve its failure prediction as well. As this paper focuses on edge and corner slab panels, Collapse Mechanism 2 is not discussed further. Fig. 2 and Fig. 4 reveal that the intermediate beams fail at an angle which leads to twisting in the beam. Including the twisting capacity into the calculation enhances the prediction by 1 min. The collapse time of edge slab panels can be calculated by either Equation 1 or 2. The notations for the equations used in this paper are explained below:

L	length of primary beam
$M_{T,pp}$	plastic sagging moment capacity of top protected primary beam at time t
$M_{B,pp}$	plastic sagging moment capacity of bottom protected primary beam at time t
$M_{L,ps}$	plastic sagging moment capacity of left protected secondary beam at time t
$M_{R,ps}$	plastic sagging moment capacity of right protected secondary beam at time t
M_{int}	plastic sagging moment capacity of unprotected intermediate beam at time t
M_i^-	plastic hogging moment capacity of each beam at the connection at time t
$b_{eff,i}$	effective width of each composite beam
l	length of secondary beam
m^+	sagging moment capacity of slab
m^-	hogging moment capacity of slab
n	number of unprotected intermediate beams
r	aspect ratio
w	applied floor load at fire limit state

Folding across right secondary beam:

$$\frac{wLl}{3}-4\left[\begin{array}{c} M_{R,ps}\frac{1}{l}+nM_{\text{int}}\frac{1}{l}+M_{L,ps}^{-}\frac{1}{l}+M_{R,ps}^{-}\frac{1}{l}+nM_{\text{int}}^{-}\frac{1}{l} \\ +\left(m^{+}+m^{-}\right)\left(L-b_{eff,L,ps}-b_{eff,R,ps}-nb_{eff,\text{int}}\right)\frac{1}{l} \\ +\left(m^{+}+m^{-}\right)\frac{l}{4L} \end{array}\right]\geq 0 \qquad (1)$$

Folding across bottom primary:

$$\frac{wLl}{3}-4\left[\begin{array}{c} M_{B,pp}\frac{1}{L}+nM_{\text{int}}\frac{1}{4l}+M_{T,pp}^{-}\frac{1}{L}+M_{B,pp}^{-}\frac{1}{L}+nM_{\text{int}}^{-}\frac{1}{4l} \\ +\left(m^{+}+m^{-}\right)\left(l-b_{eff,T,pp}-b_{eff,B,pp}\right)\frac{1}{L} \\ +\left(m^{+}+m^{-}\right)\left(L-nb_{eff,\text{int}}\right)\frac{1}{4l} \end{array}\right]\geq 0 \qquad (2)$$

It has to be mentioned that Equation 1 is for a slab panel with the right secondary beam located at the edge of a building, and Equation 2 is for a slab panel with the bottom primary beam at the edge of the building.

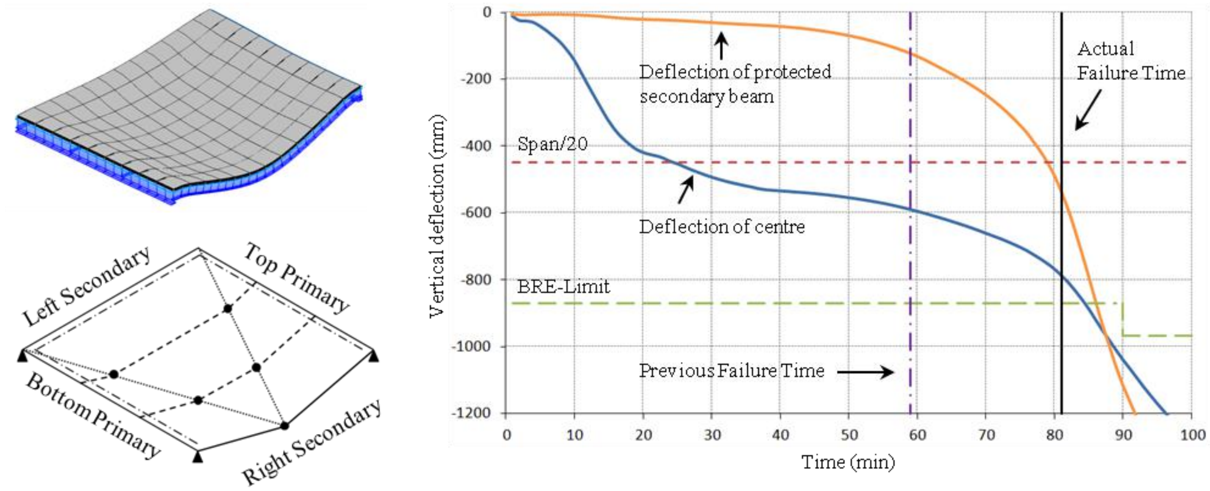

Figure 4. Collapse Mechanism 3 – Comparison

Collapse Mechanism 4

To develop a new expression for the collapse mechanism of slab panels located at the edge of a building, large-scale simulations were performed. The simulations consisted of four or six slab panels, with each panel having a size of 9 m x 9 m or 12 m x 9 m. Two main fire scenarios were explored:

1. a large fire involving all slabs in one large compartment and,
2. a local fire just beneath the corner slab panel only.

The results show that fire exposure to the whole compartment leads to structural failure across the secondary beams, which is well described by Collapse Mechanism 2. Of more interest is the simulation with only one slab panel under fire, as highlighted in Fig. 5, where the cold adjacent slab panels contribute rotation and deflection support at the internal edges of the corner slab panel. The deflected shape of these simulation also indicates simultaneous failure of the external beams. Therefore, in developing the mechanism, it was assumed that the internal edge beams maintain their vertical support throughout the fire. With this simplification the predicted collapse from the large simulation of both edge beams could be reproduced with the slab panel properties listed in Table 1.

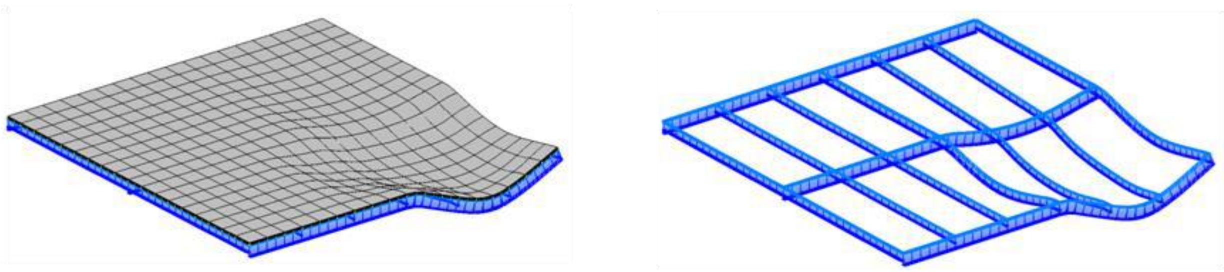

Figure 5. Fire at corner panel – Large-scale simulation

A similar collapse mechanism had been published (Park and Gamble, 2000) for reinforced concrete slabs with simple supports at two adjacent edges, without rotation capacity, and a column at the external corner. For simple concrete slabs the collapse load can be calculated by including the unknowns, which describe the position of maximum vertical deflection in the the virtual-work equations and solving their derivatives simultaneously. The present research has extended this approach to the corner slab panel. Initial investigations have yielded no logical solutions. An energy minimisation approach is thus employed. An alternative solution has been obtained by postulating the location of the maximum deflection of the slab panel, depending on the size of the slab panel, its aspect ratio and beam section properties. This was found after an extensive investigation of *Vulcan* simulations of corner slab panel failure modes. Consequently Equation 3 has been proposed for Collapse Mechanism 4. The failure mechanism depends on the aspect ratio and the length of the primary beam.

Slab panel failure occurs when:

$$\frac{r\left(375r^2+364\right)}{864\left(r^2+1\right)}wL^2-\frac{8L}{5r}\left[\begin{array}{c}\frac{12r}{5}M_{T,pp}+2M_{R,ps}+M_{\text{int}}\left(n-\frac{15n-17}{15n+5}\right)\\ +rM_{T,pp}^{-}+M_{R,ps}^{-}+nM_{\text{int}}^{-}+L\left(\frac{5r^2}{6}+\frac{25}{36}\right)m^{+}\\ +\left(m^{+}+m^{-}\right)\left[L\left(r^2+1\right)-rb_{eff,T,pp}-b_{eff,R,ps}-nb_{eff,\text{int}}\right]\end{array}\right]\geq 0 \qquad (3)$$

with aspect ratio: $r = {}^{l}\!/_{L}$

As shown in Fig. 6 this type of collapse leads to more interaction between the deflections of the primary and secondary beams. The deflection of the secondary beam is larger than the primary beam until 79 min. Thereafter the primary beam deflection exceeds that of the secondary beam. At 92 min the primary beam temporarily stops deflecting, while more load is transferred to the secondary beam, which consequently deflects rapidly. The deflection of both beams never exceeds the maximum deflection of the slab. From the *Vulcan* simulations slab panel failure is predicted at 98 min when the deflection of the slab is about 1500 mm.

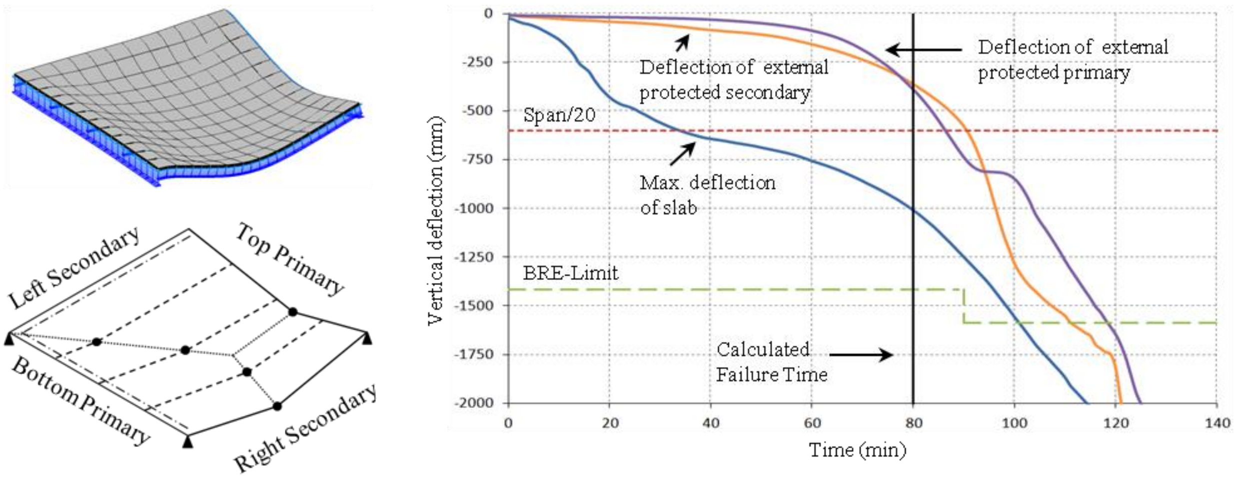

Figure 6. Collapse Mechanism 4 – Comparison

Test comparison

In recent years two full scale tests have been performed in order to investigate various aspects of tensile membrane action in composite slab panels. Both tests: CROSSFIRE (2006) and FRACOF (2008) (Zhao et al., 2008) were performed by CTICM in France. A comparison of Collapse Mechanism 4 was intended for the FRACOF-test, as it was designed to simulate the behaviour of a corner slab panel, with continuity on two adjacent edges. The slab panel size was 8.74 m x 6.6 m with two IPE400 primary beams and four IPE300 secondary beams supported by four short HEB260 columns. The slab itself was constructed with a COFRAPLUS 60 trapezoidal decking profile with a trough depth of 97 mm; overall slab thickness of 155 mm and a C30/37 normal weight concrete. During the test the load was applied by fifteen sand bags uniformly distributed over the floor to generate a uniform live load of 3.87 kN/m². Continuity was obtained by welding the reinforcement on two external steel beams. The two secondary beams in the interior and the composite slab were left unprotected whereas all boundary beams were fire protected to ensure global structure stability for a fire rating of 120 min. The floor was exposed to the standard temperature-time fire for 120 min, and then stopped due to integrity failure of the floor by a large crack in the middle of the slab.

For comparisons between the developed collapse mechanism and the FRACOF test the experimental setup was also modelled in *Vulcan*. However a reasonable comparison of the test behaviour and Collapse Mechanism 4 could not be made because no structural failure occurred in the protected edge beams, due to the very heavy protection regime. The FRACOF-test was performed primarily to demonstrate the advantages of tensile membrane action, and not to investigate potential failure mechanisms. The differences between the FRACOF slab panel and the design slab panels with which Collapse Mechanism 4 has been developed are listed in Table 2.

Table 2. Slab panel comparison

FRACOF				Design slab	
Beam Type	Load Ratio	Temperature at 60min	120min	Load Ratio	Temperature at 60min
Top Primary	0.22	110 °C	218 °C	0.42	550 °C
Left Secondary	0.27	121 °C	289 °C	0.44	547 °C
Intermediate	0.39	933 °C	1050 °C	0.45	940 °C

The load ratios of the FRACOF-beams are less than would be expected in normal beams, and the temperatures of the protected beams even after 120 min of exposure to the standard fire are below 300°C. The steel sections therefore still had 100% strength, whereas the design slab panels were designed to be as realistic as possible, with standard protection regime. It has to be mentioned again that the FRACOF-test was not designed to achieve a structural failure of the whole slab panel and therefore it is difficult to compare the test results with the developed collapse mechanism.

Conclusion

Two collapse mechanisms have been proposed to incorporate into the Bailey-BRE method to enhance its use. They include moment capacity at continuous edges and loss of vertical support at slab panel boundaries. One of the proposals, for edge panels, has been developed in recent years but was found to be deficient, so it has been improved to give better predictions of structural failure. The second proposal considers slab panels located at the corner of a building and provides a prediction of the failure time. Comparisons between this collapse mechanism and the FRACOF-test could not be made because the test concentrated on promoting tensile membrane action. Due to this the edge beams had a low load ratio and were heavily protected.

The present study concentrated on collapse mechanisms of particular slab panels. Large-scale simulations have shown that the supporting effect of adjacent slab panels, depending on their orientations, affect the resistance of the panel. These effects have been included with simplifications which may need further development. It has also been observed that the consideration of fire under one slab panel or in a large compartment influences the type of structural failure. In the study, the effects of columns were not considered. Including them into the models will provide some axial restraint to the beams and the slab panels as a whole. The supporting effect of adjacent slab panels and horizontal restraint from columns are subjects for future research.

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