Cover page

Title: Collapse Mechanisms of Composite Slab Panels in Fire

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ABSTRACT

The identification of tensile membrane action as a sustainable, high-capacity load-bearing mechanism of composite floors under fire conditions has led to the development of a number of simplified design solutions, because of the unsuitability of finite element analysis for routine design. Prominent amongst these is the Bailey-BRE method, which predicts composite slab capacity by calculating the enhancement of its traditional yield-line load capacity due to tensile membrane action. This method assumes that the two-way bending slab panel, composed internally of parallel unprotected composite beams, is supported on edges which resist vertical deflection. In practice, the protected composite beams which simulate this vertical edge support in fire deflect under the combination of heating and load, and this loss of vertical support induces single-curvature bending, which leads to an eventual structural failure by folding of the slab panel.

A simple folding mechanism, which considers the contributions of the internal unprotected beams and the protected edge beams, has been developed for isolated slab panels. In the current study the mechanism has been extended to include the reinforcement in the slab as well as its continuity across the protected edge beams. Structural failure of the panel depends on the applied loads, the relative beam sizes, their locations within the building, their arrangement in the slab panel considered, the location of the slab panel and the severity of fire exposure. These factors are considered in developing a number of collapse mechanisms as an additional check within the Bailey-BRE design method. Comparisons are made with the finite element software *Vulcan* and other acceptance criteria.

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INTRODUCTION

Recent advances in structural fire engineering have paved the way for innovation and an increased use of performance-based design, especially in steel-framed buildings. Research and observations of structural behaviour under fire conditions over the past 20 years have shown that load redistribution and large deflections of parts of the structure at the Fire Limit State are essential to the survival of the entire structure. Both accidental fires and tests on full-scale buildings have shown that designing composite floors for tensile membrane action yields considerable savings in protection costs, and structural stability is maintained by taking advantage of this real building behaviour in fire [1]. Tensile membrane action is a mechanism that produces increased load-bearing capacity in thin slabs undergoing large vertical displacements, in which radial tension in the central area of a slab induces an equilibrating peripheral ring of compression. The conditions necessary for the effective use of this mechanism are two-way bending of the slab and vertical support along all of its edges. Due to its self-equilibrating nature, horizontal edge restraint is not required to mobilise tensile membrane action.

To optimise composite floors to take advantage of this higher load capacity in structural fire engineering design, a composite floor is divided into several fire-resisting rectangular zones of low aspect ratio, called slab panels, each usually comprising a set of parallel adjacent unprotected composite beams in the interior of the panel, with edges which primarily resist vertical deflection [2]. This vertical support is usually provided by protected composite beams along all four edges, and the panels are generally set out to align with the column gridlines, as shown in Figure 1.

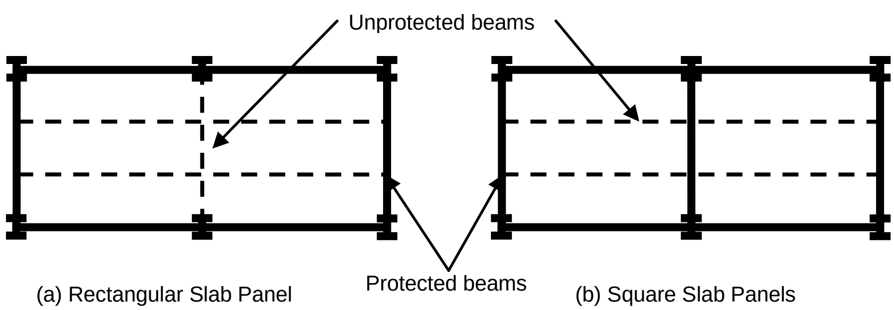

Figure 1: Rectangular and square slab panels

In fire the unprotected beams lose strength and stiffness rapidly, and their loads are then borne by the concrete slab, which undergoes two-way bending and increases its resistance as its deflections increase. At large deflections and high temperatures, the slab panel's capacity is dependent on the tensile strength of the reinforcement, provided sufficient vertical support is available at the slab panel boundary. The benefits of incorporating tensile membrane action into fire engineering design have inspired the development of several software packages to help quantify slab capacities in fire. Whole-structure behaviour in fire can be modelled in a three-dimensional framework with sophisticated finite element software (such as *Vulcan* [3], TNO DIANA, ABAQUS and SAFIR) which incorporates geometri and material nonlinearity properties of structures. Although these simulations provide useful information on complete load-deformation and stress development at elevated temperatures, they can be very costly processes; simpler methods [2, 4-6] are often preferred for routine design.

Prominent among these simplified approaches is the Bailey-BRE Method. It treats slab panels as isolated, because the large hogging moments which may be generated at the edges at large deflections are assumed to fail the slab reinforcement over the edge beams, eliminating any continuity with adjacent panels. The method analyses the panels on the assumption that the protected edge beams offer sufficient vertical restraint throughout the fire exposure. The limiting condition is the formation of a central through-depth tensile crack across the short span of the slab, which constitutes a failure of the integrity of compartmentation rather than a real structural stability failure.

However, a combination of the redistributed applied loads and long-term thermal exposure can cause significant deflections of some of the edge beams, resulting in the loss of the sustained double-curvature, which may lead to a local structural collapse. Finite element investigations of composite slab systems have confirmed this, and a simple folding mechanism has been proposed for isolated panels [7].

This paper extends the proposal to cover reinforcement, slab continuity, and to examine other potential collapse mechanisms which could be incorporated into the Bailey-BRE method to make it more robust.

COLLAPSE MECHANISMS

Structural collapse of a slab can be modelled using plastic folding mechanisms which allow collapse without generating membrane forces in the slab. is not a unified concept. It requires work-balance calculations for a range of failure modes, selecting the one which occurs first; this depends on the layout of the slab and its support conditions.

After postulating a failure mode, the algebraic expressions for external and internal work done are derived, and equating these unearths the appropriate collapse load. In fire, the load intensity is invariant at the Fire Limit State loading. However, the resistance of each structural component varies as its thermal exposure changes. For a slab panel, the overall resistance depends on the relative temperatures of its components; the slab, its reinforcement, and the protected and unprotected beams. In addition, continuity would increase the panel's capacity due to the hogging moments generated across the edge beams not involved in folding.

The development therefore calculates the reduced internal work done due to the thermal exposure at each time step, and then compares it with the 'constant' external work done for a given deflection. The point at which the internal work done ceases to exceed the external work done defines failure of the panel, and hence the failure 'time'.

Figure 2 presents a summary of the mechanisms discussed in this paper:

1. Collapse Mechanism 1 is for the isolated slab panel case.
2. Collapse Mechanism 2 follows the principles of the isolated panel, but includes continuity across two opposite sides of the slab panel. Although

uncommon, this type of failure occurs in large compartments, such as open-plan offices, where the fire can cover very large areas.

3. Collapse Mechanism 3 is appropriate to slab panels at the edge of a building, subjected to fires which are local to that compartment.
4. Collapse Mechanism 4 would cover a similar fire exposure scenario, but resulting in a different response of the slab panel, due to its location and the relative sizes of its beams.

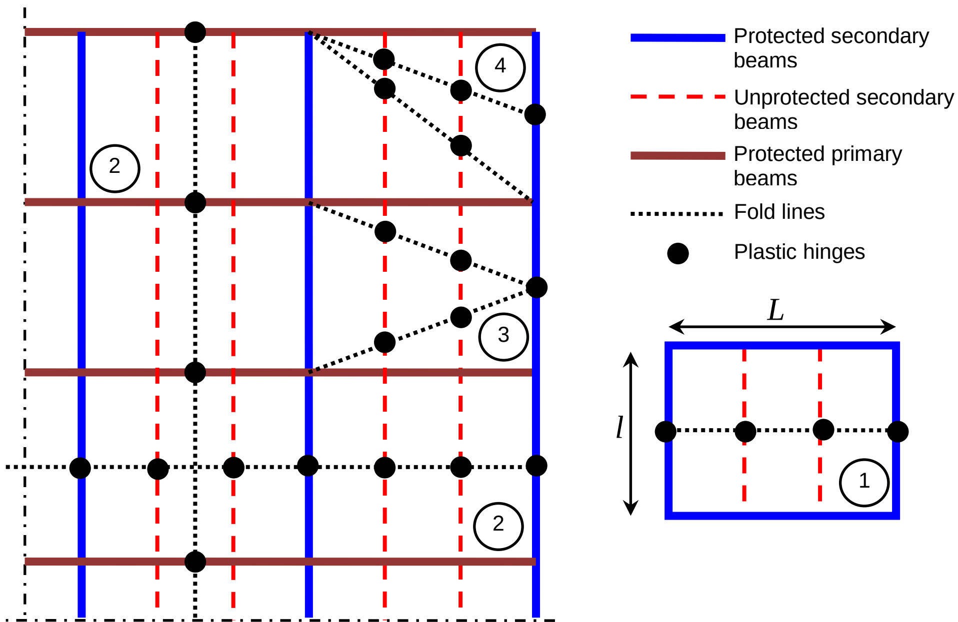

Figure 2: Proposed Collapse mechanisms

These simplified mechanisms have been verified with *Vulcan* [3], and checked against the Bailey-BRE Limit and the conventional span/20 deflection limit. The design data for the example case are: dead load = 4.33kN/m^2; live load = 5.0kN/m^2; trapezoidal decking profile with a trough depth of 60mm; overall slab thickness of 130mm, and a concrete cube strength of 40N/mm^2. The floor beams were designed to BS5950 Parts 3 and 8, and the edge beams were protected to reach a maximum temperature of 550°C at 60min Standard Fire exposure.

Two slab panel sizes (9m x 9m and 12m x 9m, with properties listed in Table 1) were used for the verification. Intermediate secondary beams were spaced at 3m.

Table 1: Slab panel design data

Slab Panel Size	Beam Type	Beam Section	Load Ratio	Temperature at 60 minutes	Span (m)
9m x 9m	Intermediate	305 x 127 x 48 UB	0.471	940°C	9
	Secondary	356 x 171 x 67 UB	0.442	550°C	9
	Primary	533 x 210 x 101 UB	0.446	548°C	9
12m x 9m	Intermediate	457 x 152 x 67 UB	0.470	941°C	9
	Secondary 1	406 x 178 x 67 UB	0.443	548°C	9
	Secondary 2	533 x 210 x 101 UB	0469	550°C	9
	Primary	610 x 305 x 179 UB	0.471	547°C	12

In the equations that follow, the following notation is used:

L	length of primary beam
l	length of secondary beam
w	applied floor load at the fire limit state
w_{eff}	effective width of composite beam
δ	maximum deflection of the slab (or beams)
θ	beam or slab rotation
M_{pp}	plastic moment capacity of the protected primary beam at time t
M_{ps}	plastic moment capacity of the protected secondary beam at time t
M_u	plastic moment capacity of the unprotected beam at time t
m^+	sagging moment capacity of the slab
m^-	hogging moment capacity of the slab
n	number of intermediate unprotected beams in the slab panel

Collapse Mechanism 1 (see Figure 2)

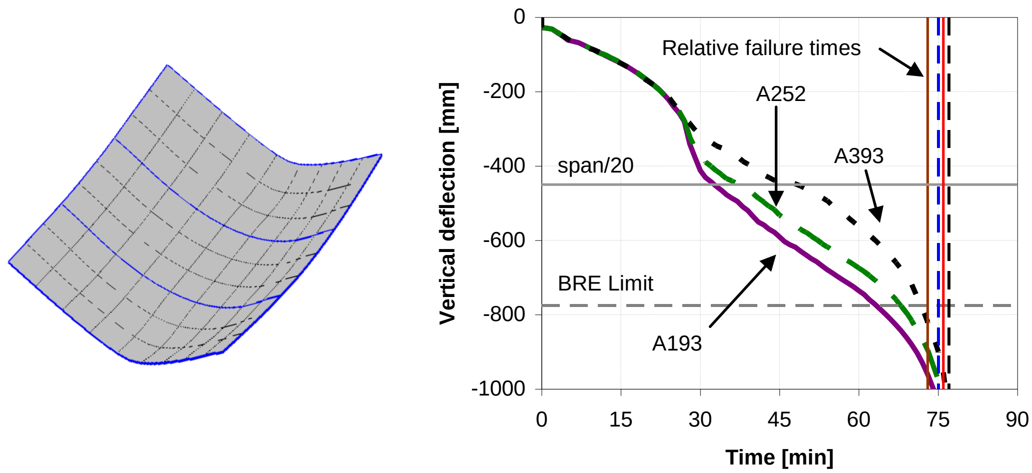

Figure 3: Collapse mechanism 1 - comparisons

Folding across secondary beams:

$$\frac{wLl}{2} - \left[8M_{ps}\frac{1}{l} + 4nM_u\frac{1}{l} + 4m^+\frac{1}{l}\left[L - (n+1)w_{eff}\right]\right] \geq 0 \qquad (1)$$

Folding across primary beams:

$$\frac{wLl}{2} - \left[8M_{pp}\frac{1}{L} + 4m^+\frac{1}{L}l\right] \geq 0 \qquad (2)$$

In the equations above, the plastic capacities of the composite beams include the contribution of the reinforcement in the slab which forms the upper flange. The term (L-(n+1)w_{eff}) therefore accounts for the parts of the reinforced concrete slab which do not form parts of the effective upper flange widths.

Figure 3 shows a snapshot from the *Vulcan* analysis of the single-curvature folding of the isolated 9m x 9m slab panel. Following from the previous study [7] the analysis was conducted with 3 mesh sizes – A193, A252 and A393 (193, 252 and 393 mm^2/m in each direction, respectively). It is observed that the mesh size influences the failure time – a feature which is picked up by the improved proposal. The 4 vertical lines in the plot signify the failure times without reinforcement (73min), with A193 (75min), A252 (76min) and A393 (77min) respectively from left to right.

Collapse Mechanism 2 (see Figure 2)

Failure across secondary beams:

$$\frac{wLl}{2} - \left[8M_{ps}\frac{1}{l} + 4nM_u\frac{1}{l} + 4m^+\frac{1}{l}\left[L-(n+1)w_{eff}\right] + 4m^-\frac{1}{l}L \right] \geq 0 \tag{3}$$

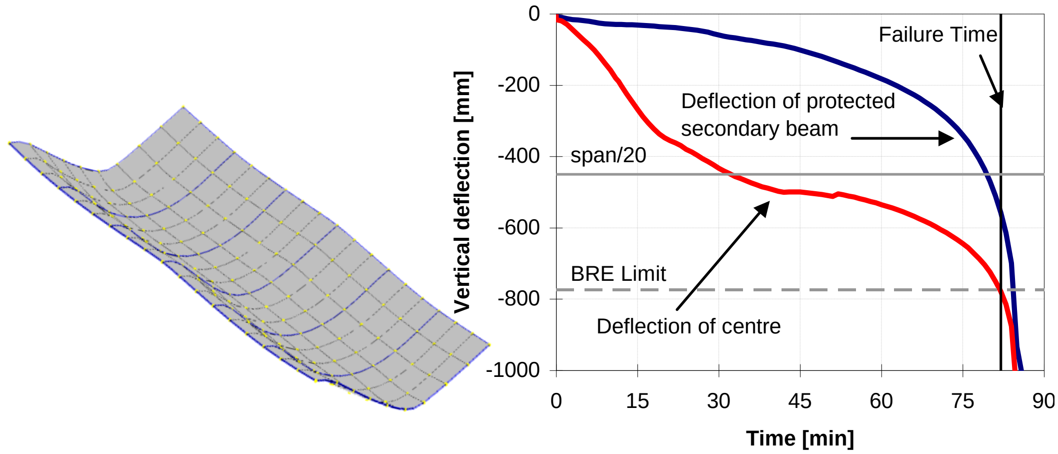

Figure 4: Collapse Mechanism 2 - comparisons

Failure across primary beams:

$$\frac{wLl}{2} - \left[8M_{pp}\frac{1}{L} + 4\left(m^+ + m^-\right)\frac{1}{L}l \right] \geq 0 \tag{4}$$

Assumptions made in the determination of hogging moments due to the reinforcement are:

1. The rebar breaks prematurely (the Bailey assumption), which results in either isolated slab panels or a free-edge condition.
2. The net compressive force acts at the centroid of the connection (possibly the beam centroid if primary and secondary beams have the same cross-section). For the analyses discussed here it is assumed that the net compression is at the centroid of the unprotected steel I-beam.
3. The bottom flange of the secondary beam contacts the web of the primary beam.

A verification of Collapse Mechanism 2 is shown in Figure 4. The *Vulcan* analysis on the left is of two bays of 9m x 9m slab panels, with continuity across one edge of primary beams. This simulates a fire in a large compartment in the outer bays of a building. The graph on the right shows deflections of the centre of the 9m x 9m slab panel and the protected secondary beam between the two panels. It is observed that, due to the redistribution of loads that takes place, coupled with thermal exposure, the protected beam deflection 'runs away', resulting in another single-curvature failure. The proposed collapse mechanism is seen to give a good approximation of the failure time (82min) of the multi-bay model.

Collapse Mechanism 3 (see Figure 2)

Failure of protected secondary beam:

$$\frac{wLl}{3} - \left[\begin{array}{l} 4M_{ps}\dfrac{1}{l} + 4nM_u\dfrac{1}{l} + 4m^+\dfrac{1}{l}\left[L-(n+1)w_{eff}\right] \\ +4m^-\dfrac{1}{l}L + 2\left(m^+ + m^-\right)\dfrac{l}{L} \end{array} \right] \geq 0 \qquad (5)$$

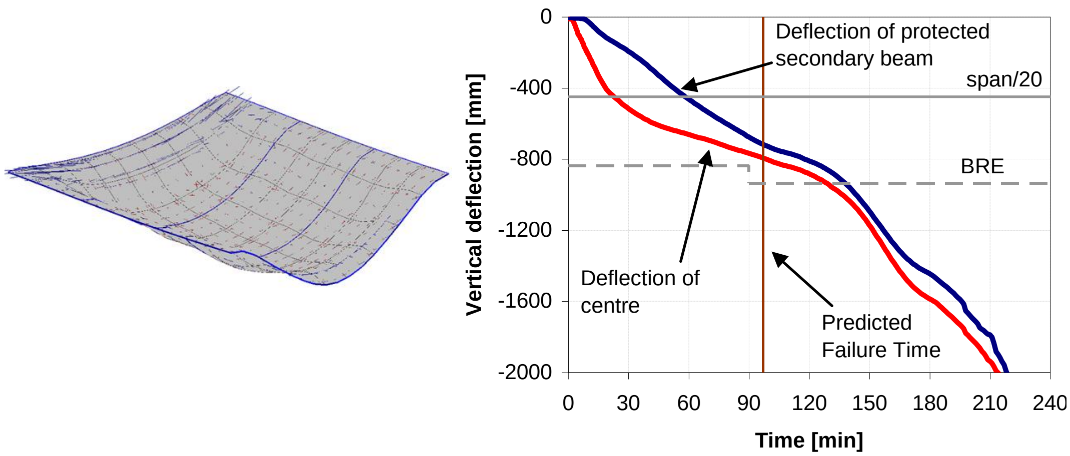

Figure 5: Collapse Mechanism 3 - comparisons

Failure of protected primary beam:

$$\frac{wLl}{3} - \left[\begin{array}{l} 4M_{pp}\dfrac{1}{L} + 4\left(m^+ + m^-\right)\dfrac{1}{L}l + m^-\dfrac{L}{l} \\ +m^+\left[L-(n+1)w_{eff}\right]\dfrac{1}{l} \end{array} \right] \geq 0 \qquad (6)$$

A comparison of deflections of the 12m x 9m *Vulcan* model and the limiting deflections is shown in Figure 5. Relative to the span/20 criterion, the slab panel fails at about 25min, while the Bailey-BRE limit suggests the panel can adequately survive a 120min exposure to the standard fire. However, the deflection-time plot shows no clear sign of failure until about 150min into the fire when the deflections of the centres of both the panel and the edge beam accelerate. The proposed collapse mechanism gives a very conservative prediction. In the derivations so far, the fold lines have been perpendicular to the orientation of either the failing secondary or primary beams. In this particular case, they are at an angle, and could especially influence the capacities of the unprotected beams. The *Vulcan* models show a twisting of the unprotected beams at the plastic hinges; the next update of these collapse mechanisms will explore the possibility of incorporating this in the method to improve predictions.

Collapse Mechanism 4 (see Figure 2)

Collapse Mechanism 4 is proposed to predict the failure of corner panels. The aim is to determine the influence of different support conditions and beam sizes on slab panel failure. The suggested failure mechanism is shown in Figure 2. However, comparisons using the numerical models has shown failure modes similar to those of isolated slab panels, folding in single-curvature directly across secondary beams, regardless of the support conditions at the edges of the panel. More investigation will be carried out to ascertain this and evaluate how aspect ratios influence this behaviour.

CONCLUSION

A number of collapse mechanisms have been proposed for inclusion into the Bailey-BRE Method to act as an extra check against structural collapse, as distinct from the compartment integrity failure on which the existing method is based. Two of the proposals, for isolated panels and for large compartments, give accurate predictions, but those for edge panels and panels in the corner of a building require improvement.

In the study, the effects of columns were not considered. The presence of columns will provide some axial restraint to the beams and the slab panels as a whole. Depending on the loads on the beams attached to the columns and the columns themselves, coupled with fire severity, the beams could fail, pulling the columns inwards which could potentially fail due to p-Δ effects. This is however a subject for future research.

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