

THE EFFECT OF EDGE SUPPORT ON TENSILE MEMBRANE ACTION OF COMPOSITE SLABS IN FIRE

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Abstract. *The Bailey-BRE Method is a simplified design approach that facilitates the use of a tensile membrane action design philosophy for composite floors under fire conditions. The method requires the division of a composite floor into rectangular slab panels, composed of parallel unprotected composite beams in their interior parts, supported vertically by protected composite edge beams. Enhanced slab capacities are obtained after the unprotected beams have lost significant strength, by allowing large deflections of the slab in biaxial bending. The use of tensile membrane action generates significant cost savings in composite structures, as a large number of floor beams can be left unprotected. However, the protected beams which provide vertical support to the edges of panels lose strength under the combined effects of thermal degradation and the increased loading due to biaxial bending, and this has the potential to cause panels to lose structural stability altogether. It is therefore imperative to investigate what constitutes adequate vertical support and the detrimental effects of inadequate vertical support on tensile membrane action of composite slabs in fire. This paper reports on a study of this effect, and puts forward some simple recommendations to avoid loss of stability of composite floors designed by this method.*

1 INTRODUCTION

Improved understanding of real structural behaviour under fire conditions in recent years has resulted in the increased use of performance-based design methods to ensure the fire resistance of steel structures. Traditional methods, in which all exposed steelwork is protected in the aftermath of normal limit-state design which ignores the fire case, are being replaced by performance-based alternatives, which ensure structural stability and in many cases offer considerable savings. In particular, it has been observed that, if composite floors are allowed to undergo large vertical displacements in biaxial bending during heating by fire, they can achieve load-bearing capacities several times greater than their traditional yield-line capacities, through a mechanism known as tensile membrane action [1]. This mechanism produces increased load-bearing capacity, particularly in thin slabs undergoing large vertical displacements. It is characterised by a large area of radial tension in the central area of a slab which induces an equilibrating peripheral ring of compression. The conditions necessary for this mechanism to be effective 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 for the mobilisation of tensile membrane action, in contrast to the catenary tension which can occur in one-way bending.

To optimise composite floors to take advantage of this enhanced load capacity in structural fire engineering design, a composite floor is divided into a number of fire-resisting rectangular areas, preferably of low aspect ratio, called slab panels. Each of these comprises one or more parallel unprotected composite beams in the interior part of the panel, with edges whose role is primarily to resist

vertical deflection. This vertical support is usually provided by thermally-protected primary and secondary composite beams along all four edges, and these are generally directly supported by columns on the building's main gridlines, as shown in Figure 1. Composite slabs are usually reinforced with light meshes (typically with steel areas between $142\text{mm}^2/\text{m}$ and $393\text{mm}^2/\text{m}$) whose normal role is to control cracking during construction. In fire the unprotected beams lose strength and stiffness rapidly, and their loads are then borne by the composite slab, which undergoes two-way bending and increases its load capacity as its deflections increase.

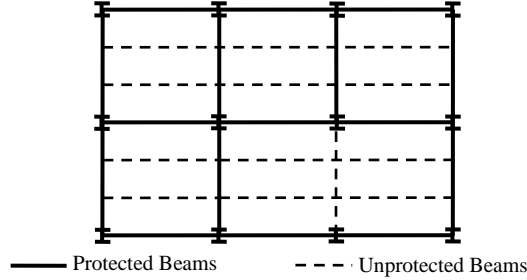


Figure 1: Typical slab panels

At large deflections and high temperatures, a slab panel's capacity is dependent on the tensile capacity of the reinforcement, provided that adequate vertical support is available at its boundary. The merits of incorporating tensile membrane action into structural fire engineering design have prompted the development of several software packages to help quantify slab capacities in fire. Tensile membrane action, as a part of whole-structure behaviour at high temperatures, can be modelled in a three-dimensional framework with sophisticated finite element software, such as *Vulcan* [2]-[3], TNO DIANA, SAFIR and ABAQUS, which incorporate geometrically nonlinear effects of structures as well as nonlinear material behaviour. Although such finite element simulations provide useful information on complete load-deformation behaviour and stress development at elevated temperatures, they can be very costly processes in computational effort and in runtimes. Simpler performance-based methods, such as the Bailey-BRE membrane action method [1] or the New Zealand Slab Panel Method [4] (which can easily be set up as a spreadsheet), are often preferred for routine design. However, there have been some suspicions that the simplifications applied in these approaches can lead to unrealistic or over-conservative designs.

In order to assess their efficiency as tools for preliminary investigations, there is an implicit need to determine the limitations of these simplified methods. The reliance of the Bailey-BRE method on the determination of enhancements to the traditional yield-line capacity of the slab, the assumption of continuous vertical support throughout the duration of a fire, and the detrimental effects of structural failure of the edge beams are some of the issues that need to be addressed. The study reported here initially examined the credibility of the Bailey-BRE method through the use of a finite element study, with the aim of establishing slab panel capacities as a function of the amount of reinforcement within a panel and the degree of vertical support available along the slab panel boundary.

2 THE BAILEY-BRE METHOD & TSLAB

Tensile membrane action (TMA) was observed as a primary load-bearing mechanism of composite slabs in fire after a series of full-scale fire tests in the United Kingdom in the 1990s [5]. The Bailey-BRE method was developed in order to simplify the process of incorporating the rather complex mechanics of TMA into routine design of composite slabs in fire. The method divides a composite floor into several rectangular slab panels of low aspect ratio. Based on a conservative assumption that the light slab reinforcement will fracture in hogging over protected beams when the composite slabs are continuous, the

Bailey-BRE method [1, 6] treats each slab panel as isolated in the sense that it is horizontally unrestrained, but as vertically supported along its edges (see Figure 2). Each of these panels is composed internally of simply supported unprotected composite beams, spanning in only one direction. With increasing exposure to elevated temperatures, the formation of plastic hinges in the unprotected beams redistributes their loads to the biaxially bending slab, undergoing large vertical deflections. By employing rigid-plastic theory with large change of geometry, the additional slab resistance provided by tensile membrane action is calculated as an enhancement to the small-deflection yield-line slab capacity; the enhancement increases with increasing slab deflection. Failure of the integrity of the slab as a separating element is determined by the eventual formation of a full-depth tension crack across the shorter span of the slab; in highly-reinforced slabs there may alternatively be a compressive failure of concrete at the corners.

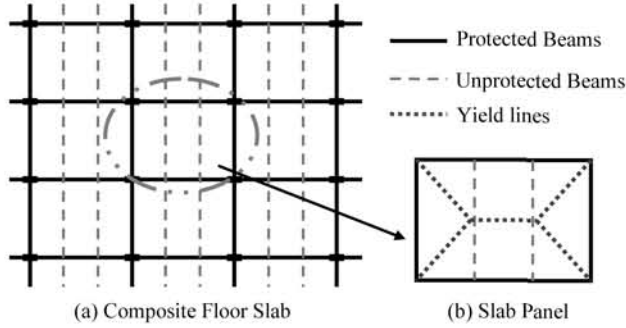


Figure 2: Schematic diagram of the Bailey-BRE Method

The protected beams around the edges of slab panels are assessed as isolated beams under their increased load ratios due to load-shedding from the unprotected beams. The method conservatively ignores any contribution of the tensile strength of concrete to the capacity of the slab, and does not provide any additional information on the state of the protected boundary beams, apart from the assumption that they remain vertically undeflected throughout a fire. The procedure, developed as an extension of the principles of the ambient-temperature behaviour to the elevated-temperature phase, assumes that yield-lines formed at small deflections are maintained at elevated temperatures [1]. However, subsequent research has shown that at elevated temperatures the differential thermal expansion through the depth of the slab thickness induces deflections sufficient to induce tensile membrane action, and that yield line patterns form in the plane of the slab when significant reductions in material strength have occurred and failure is imminent [7]-[8].

To facilitate the use of the Bailey-BRE method in the United Kingdom, a design guide [5] and the spreadsheet-based software TSLAB have been produced. Using the generic version of Bailey-BRE, a particular slab panel arrangement (a vertically-supported rectangular reinforced-concrete slab incorporating unprotected secondary beams) can be optimised for a given fire limit state, based on an allowable vertical deflection limit, expressions for which are shown in Equations 1 and 2. This limit is based on the mechanical strain allowed in the reinforcement at yield combined with thermal bowing in the slab under an assumed linear temperature gradient. In these equations, α is the coefficient of thermal expansion; 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 and f_y and E are respectively the yield strength and Young's modulus of the reinforcing steel at ambient temperature.

$$v = \frac{\alpha(T_2 - T_1)l^2}{19.2h} + \sqrt{\frac{0.5f_y}{E}} \times \frac{3L^2}{8} \quad (1)$$

$t=20^\circ\text{C}$

$$v \leq \frac{\alpha(T_2 - T_1)l^2}{19.2h} + \frac{l}{30} \quad (2)$$

TSLAB begins by performing thermal analyses on the unprotected intermediate beam and the composite slab. Then, using the temperatures of the individual components and an allowable vertical deflection criterion, it calculates the total capacity of the simply-supported slab panel model by summation of the residual capacity of the unprotected beams and the enhanced slab capacity. This capacity is then checked against the applied load at the Fire Limit State. If the capacity of the panel is found to be below the applied load at the fire limit state. This determines whether either the capacity of the internal beams or the reinforcement mesh size needs to be increased.

The Bailey-BRE method has previously been compared with fundamental approaches based on finite element analyses. An investigation by Huang *et al.* [9] into the effects of a panel's horizontal edge support conditions revealed that the Bailey-BRE method correlated very closely with a hinge-supported slab (allowing no pull-in at the edges), although it had been developed on the basis of simple supports. Another investigation into the effects of increased reinforcement ratios on slab panel capacity showed that only a marginal increase in slab panel resistance was observed in finite element models with an aspect ratio of 1.0, while disproportionately large increases in strength were observed in the Bailey-BRE models [10, 11]. It was also observed that the finite element models compared closely with the Bailey approach when high reinforcement ratios were used in slabs of aspect ratio 2.0. The observations led Foster [7] and Bailey and Toh [6] to perform experimental tests on small-scale slabs at ambient and elevated temperatures. They examined slabs with various reinforcement ratios with varying rebar ductilities. The experiments showed that high reinforcement ratios could cause compressive failure of concrete in the slab corners, and the Bailey method was modified accordingly [6]. The Bailey-BRE method determines its slab capacities by calculating the enhancements to the theoretical yield-line capacity provided by large deflections. This suggests that increasing reinforcement diameter increases the capacity of the slabs, since the yield-line capacities will themselves be considerably increased. Therefore, with a given enhancement from large deflections, a considerable slab capacity can be obtained by applying modest increases in reinforcement area. Composite slabs are normally lightly reinforced to control cracking during construction, and therefore, may fail in compression if they are over-reinforced.

In practice, slab panel vertical support is achieved by protecting the beams around the perimeter of each panel. The assumption of continuous vertical restraint at all times during a fire must therefore be unrealistic. During the fire, the combination of imposed loads, together with loss of strength and stiffness of the perimeter beams, will progressively induce vertical displacements which will reduce curvatures in at least one direction, affecting the generation of tensile membrane action. Eventually the reduction of strength of the supporting beams will allow the formation of a single-curvature slab-bending (folding) mechanism. The slab panel will then impose horizontal tying-force components on its connections, in addition to the vertical forces for which they are designed. Depending on the location of the panel, this may lead either to pull-in of columns or to connection failure, both of which are real structural resistance failures, in contrast to the compartment integrity failure which is used as the normal limiting condition. The potential for these additional modes of failure has led to the series of finite element studies reported here, into the effects of reinforcement and slab panel vertical support on composite slab behaviour in fire.

3 SLAB PANEL ANALYSES

The three slab panel layouts shown in Figure 3 were used for the structural analyses. The 9m x 6m, 9m x 9m and 9m x 12m panels were designed for 60 minutes' standard fire resistance, assuming normal-weight concrete of cube strength 40MPa and a characteristic imposed load on the slab of 5.0kN/m², plus 1.7kN/m² for ceilings and services. Using the trapezoidal slab profile shown in Figure 4, the requirements of SCI P-288 [5] and the slab specifications given in Table 1, the floor beams were designed according to BS5950-3 [12] and BS5950-8 [13], assuming full composite action between steel and concrete and simple support to all beams, in line with common UK engineering practice. The "Office"

usage class was assumed, so that the partial safety factors applied were 1.4 (dead) and 1.6 (imposed) for ULS and 1.0 and 0.5 for FLS. The assumed uniform cross-section temperatures of the protected beams were limited to 550°C at 60 minutes. The ambient- and elevated-temperature designs resulted in specification of the steel beam sizes shown in Table 2.

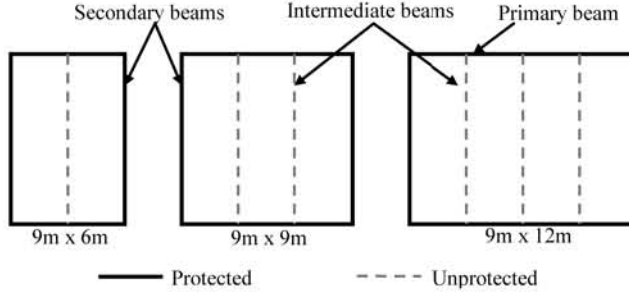


Figure 3: Slab Panel Sizes

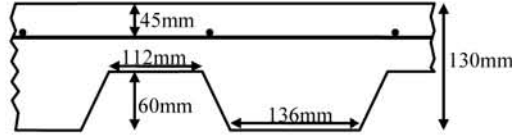


Figure 4: Concrete slab cross-section

Table 1: Slab panel requirements (R60)

Slab Panel size	9m x 6m	9m x 9m	9m x 12m
Dead load (kN/m ²)	4.33	4.33	4.33
Live load (kN/m ²)	5.0	5.0	5.0
Additional load (kN)	14	37	49
Beam design factor	0.77	1.00	0.83
Min. Mesh size	A193	A193	A252

Table 2: Protected beam design data (R60)

Slab Panel Size	Beam Type	Beam Section	Load Ratio	Limiting Temperature	Temperature at 60 minutes
9m x 6m	Secondary	356 x 171 x 57 UB	0.426	636°C	548°C
	Primary	406 x 378 x 60 UB	0.452	627°C	549°C
9m x 9m	Secondary	356 x 171 x 67 UB	0.442	630°C	550°C
	Primary	533 x 210 x 101 UB	0.446	629°C	548°C
9m x 12m	Secondary	406 x 178 x 67 UB	0.447	629°C	548°C
	Primary	610 x 305 x 179 UB	0.471	620°C	547°C

The assessment in this paper is presented as a comparison between the Bailey-BRE method and *Vulcan* finite element analysis. Both the Bailey-BRE Method and TSLAB implicitly assume that the edges of a slab panel do not deflect vertically. The progressive loss of strength of the intermediate unprotected beams is captured by a reduction in the steel yield stress with temperature. The reduced capacity of the unprotected composite beams is deducted from the total applied load at the fire limit state

to determine the load which needs to be carried by TMA, and by implication the vertical displacement required by the reinforced concrete slab (whose yield-line capacity also reduces with temperature) to generate sufficient enhancement to carry this load. The required displacement is then limited by applying Equations 1 and 2. The *Vulcan* finite element analysis, on the other hand, properly models the behaviour of protected edge beams, with full vertical support available only at the corners of each panel. *Vulcan* is a three-dimensional geometrically-nonlinear specialised finite element program which also considers nonlinear elevated-temperature material behaviour [2, 3] capable of modelling both membrane and bending effects in slabs and beams.

The analyses were initially performed with the standard isotropic reinforcing mesh sizes A142, A193, A252 and A393. These are respectively composed of 6mm-, 7mm-, 8mm- and 10mm-diameter bars of 500N/mm² yield strength at 200mm spacing in either direction. The structural properties of the two models were selected to be consistent with the assumptions of the Bailey-BRE Method [1, 6]. The required vertical displacements of the Bailey-BRE approach and the central vertical deflections of the *Vulcan* analyses have been compared with the TSLAB, BRE and Standard Fire Test (I/20) deflection limits. The results are also compared with a simple slab panel failure mechanism [8] in Figure 5. This mechanism determines the time at which the slab panel loses tensile membrane action, and goes into single-curvature bending. Using a plastic energy-work balance equation, it predicts when the parallel arrangements of primary or secondary (unprotected intermediate and protected secondary) composite beams simultaneously lose their ability to carry the applied fire limit state load due to their temperature-induced strength reductions.

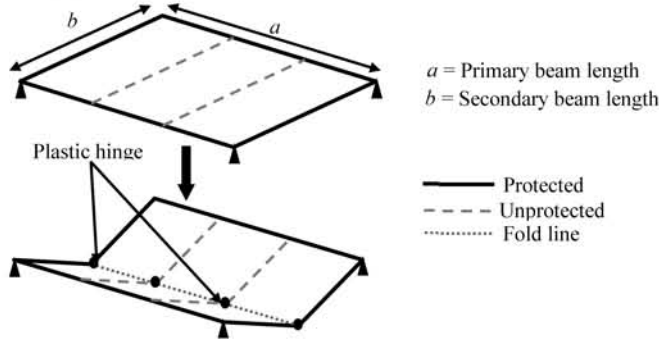


Figure 5: Slab panel failure mechanism

The expressions for failure of either primary or secondary beams are shown in Equations (3) and (4) respectively as:

Primary beam failure

$$\frac{wab}{2} - \frac{4\sum M_p}{a} \geq 0 \quad (3)$$

Secondary beam failure

$$\frac{wab}{2} - \left(\frac{4\sum M_s}{b} + \frac{4\sum M_u}{b} \right) \geq 0 \quad (4)$$

In the equations above a and b are the lengths of the primary and secondary beams; w is the applied fire limit state floor loading and M_u , M_s and M_p are the temperature-dependent capacities of the unprotected, protected secondary and protected primary composite beams, respectively, at the temperatures corresponding to any given time.

The observations from these early analyses led to a more detailed investigation of the combined effects of edge-beam stability and the reinforcement ratios on slab panel failure in fire. For like-against-like comparison of the Bailey-BRE Method and *Vulcan* analyses, the temperature profile of slab panels which is used by TSLAB was adopted. The unprotected intermediate beam temperatures from TSLAB were applied directly to the two models. TSLAB generates weighted mean temperatures of the slab top surface, bottom surface and reinforcement. These were applied directly to the Bailey-BRE models. The same could not be assumed for the *Vulcan* analyses, as fictitious temperatures had to be assumed for the other layers in the slab's cross-section. These assumptions could potentially adversely influence both thermal and stress-related strains in the model. Thus, following the earlier research [8], a one-dimensional thermal analysis of the average depth of the profiled slab (100mm) was performed with the software FPRCBC-T [14]. The temperatures generated in this way correlated very closely with those from TSLAB, and were applied in the analyses.

4 RESULTS

The results of the comparative analyses shown in Figures 6-8 show slab panel deflections for different reinforcement mesh sizes. For ease of comparison, the A142-reinforced panels are shown as dotted lines, while those reinforced with A193, A252 and A393 are shown as dashed, solid and chain-dot lines respectively. For clarity the two slab panel types are shown on separate graphs ('a' and 'b') for the Bailey-BRE Method and the *Vulcan* analyses, respectively. These differ, in that graphs 'a' show the vertical displacements required by the Bailey-BRE Method and 'b' show actual vertical deflections from the *Vulcan* analyses. The limiting deflections and the times at which plastic folding of the slab, including the protected edge beams, takes place are also shown. Regardless of the layout of a panel, it was observed that the single-curvature fold line always occurred first across secondary beams; the associated collapse times are indicated by the vertical lines in the figures. The temperatures of the various intermediate and protected secondary beams at failure are shown in Table 3 for the three slab panel layouts. Apart from the 9m x 6m panel it can be seen that failure occurred when the protected secondary beams were below their own limiting temperatures (see Table 2).

Table 3: Slab panel failure times and corresponding secondary beam temperatures (R60)

Slab Panel	Failure time	Intermediate beam temperature	Secondary beam temperature
9m x 6m	82min	983°C	663°C
9m x 9m	73min	963°C	621°C
9m x 12m	68min	952°C	594°C

4.1 Slab panel analyses

SCI P-288 [5] specifies A193 as the minimum reinforcing mesh required for 60 minutes' fire resistance. Figure 6a shows the required Bailey-BRE displacements together with the deflection limits and the slab panel collapse time. A193 mesh satisfies the BRE limit, but is inadequate for 60 minutes' fire resistance according to TSLAB. A252 and A393 satisfy all deflection criteria. It should be noted that there is no indication of failure of the panels according to Bailey-BRE, even when the collapse time is approached. This is partly due to the behaviour of the edge beams being neglected; runaway failure of Bailey-BRE panels is only evident in the required deflections when the reinforcement has lost significant strength. *Vulcan* deflections are shown in Figure 6b, and it can be seen that the panel with A393 mesh just satisfies the BRE limiting deflection at 60 minutes. It can also be seen that the deflections of the various *Vulcan* analyses converge at the 'collapse time' (82min) given by the simple slab panel folding mechanism. This clearly indicates the loss of bending capacity of the protected secondary beams. Comparing Figures 6a and 6b, the Bailey-BRE Method predicts substantial enhancement of the panel fire resistance with increasing reinforcement mesh size, while *Vulcan* shows a marginal increase. The Bailey-

BRE approach is also found to be conservative with A142 and A193 and unconservative with the larger mesh sizes.

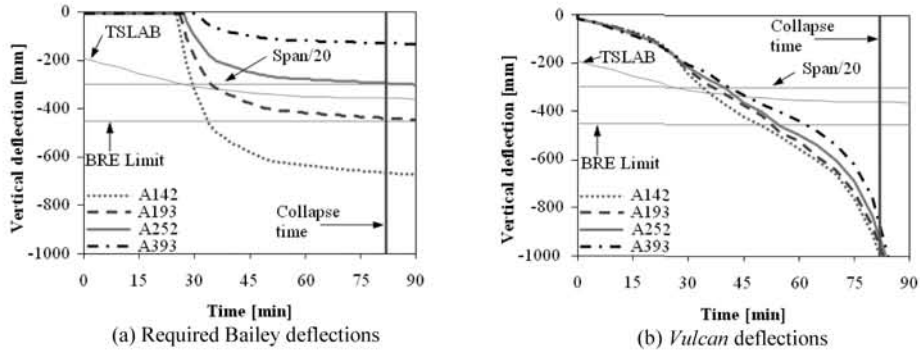


Figure 6: 9m x 6m Slab Panel results

In the 9m x 6m slab panel the secondary beams were longer than the primary beams, but in the 9m x 12m layout this is reversed. However, the large overall size of the 9m x 12m panel requires its minimum mesh size to be A252 [5]. From the required displacement plots in Figure 7a, A252 satisfies a 60-minute fire resistance requirement with respect to the Bailey-BRE limit. It is observed that increasing the mesh size from A252 to A393 results in an increase in the slab panel capacity from about 37min to over 90min, according to the TSLAB deflection limit. The same cannot be said for the *Vulcan* results (Figure 7b), which show very little increase in capacity with larger meshes.

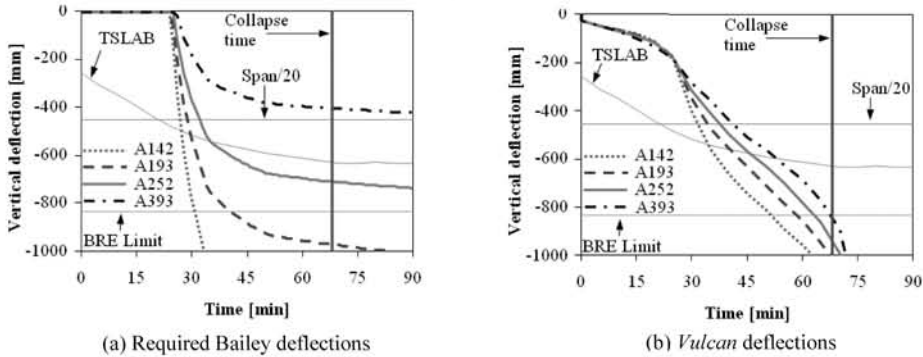


Figure 7: 9m x 12m Slab Panel results

It is shown that A252 and A393 meshes meet the fire resistance requirement at 60 minutes with respect to the BRE limiting deflection. It is also observed that the *Vulcan* deflections appear to converge on the slab panel folding collapse time of 68min. At failure, the protected secondary beams are at 594°C, which is considerably below their limiting temperature. In this study, sufficient protection was applied to all protected beams to ensure that their common design temperature (at 60 minutes) was limited to 550°C. Typically, each gridline beam would be protected to achieve a temperature just below its critical temperature at the required fire resistance time, in order to save cost. Since these temperatures would all be above 550°C this would potentially cause structural failure of the panel earlier than 68min.

Figure 8 shows results for the 9m x 9m slab panel, plotted together with the edge beam folding collapse mechanism and the three deflection criteria. The discrepancy between the Bailey-BRE limit and

TSLAB is evident once again; the recommended minimum reinforcement for 60 minutes' fire resistance, A193, is adequate with respect to the BRE limit, but fails with the TSLAB limit. As reported for the other panel layouts, an increase in mesh size results in a disproportionately large increase in the Bailey-BRE panel resistance (Figure 8a) while *Vulcan* shows a much more modest increase. Failure of the protected secondary beams at 73min limits any contribution the reinforcement might have made to the panel's capacity.

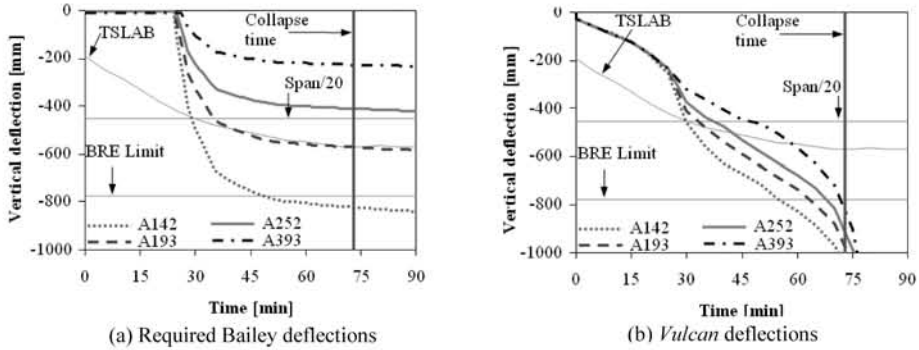


Figure 8: 9m x 9m Slab Panel results

The comparisons shown in Figures 6-8 show that the finite element method indicates marginal increases in slab panel capacity with increasing reinforcement size. The Bailey-BRE method, on the other hand, shows huge gains in slab panel resistance with larger mesh sizes. This is true even when the comparison is made in terms of the relative displacements given by the finite element analyses, although this has not been plotted here. Results for the 9m x 6m and 9m x 9m slab panels have shown that the Bailey-BRE method is conservative with the lower reinforcement sizes, while higher mesh sizes cause an overestimate of slab panel capacities. The 9m x 12m panel, however, requires higher reinforcement sizes. The *Vulcan* results show that slab panel capacity is affected much more directly by geometry than by reinforcement area. There is a need to incorporate the effect of edge beams into the simplified Bailey-BRE analysis, and so the results of a more detailed study of the effect of reinforcement area relative to slab panel failure are now presented.

3.2 Effects of reinforcement ratios

The comparison in the previous section showed that the Bailey-BRE Method can predict very high increases of slab panel capacity from small changes in reinforcement area, while *Vulcan* on the other hand indicates only marginal increases. Ignoring the structural response of the protected secondary beams seems to be the key to this over-optimistic prediction by the Bailey-BRE Method. Therefore, to investigate the real contribution of reinforcement ratios, structural failure of the panel as a whole by plastic folding has been incorporated as a further limit on the Bailey-BRE deflection range. Fictitious intermediate reinforcement sizes have been used in addition to the standard meshes, in order to investigate in a more continuous fashion the effects of increasing reinforcement area on slab panel resistance. The range of reinforcement area is maintained between 142mm²/m and 393mm²/m, with additional areas of 166, 221, 284, 318 and 354 (mm²/m). The investigation in this section examines failure times of the slab panel with respect to the three deflection criteria (TSLAB, BRE Limit and Span/20) normalised with respect to the time to creation of a panel folding mechanism, since this is a real structural collapse of the entire slab panel. Results for the 9m x 6m, 9m x 12m and 9m x 9m panels are shown in Figure 9. The lightly-shaded solid curves show results from the Bailey-BRE Method. Those from *Vulcan* are shown as darker solid curves. The dotted, solid and dashed lines refer respectively to failure times with respect to the (short span/20) criterion, the TSLAB deflection limit and the BRE limit.

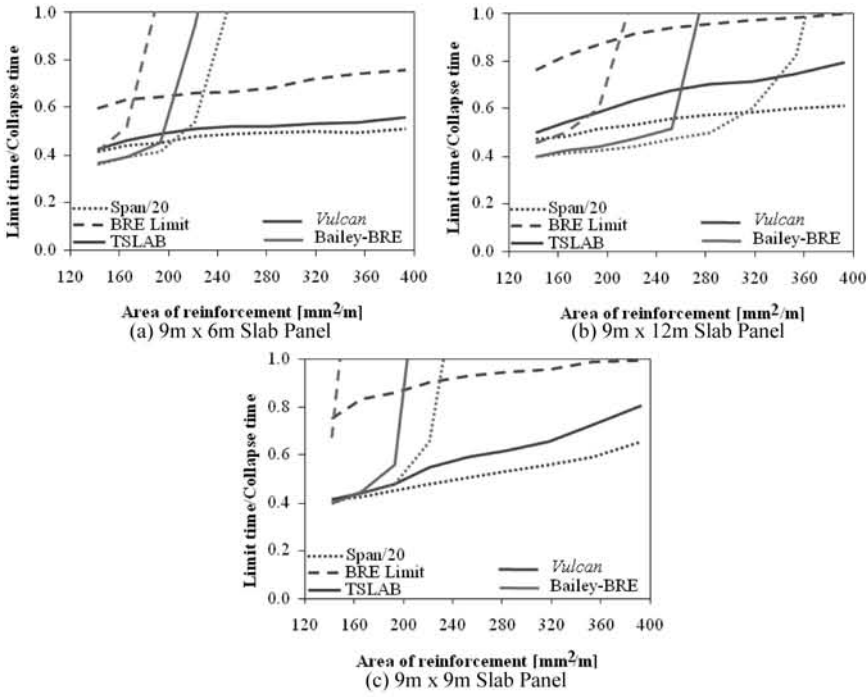


Figure 9: Normalised slab panel failure times

The comparisons in Figure 9 further confirm that the Bailey-BRE Method is conservative for the lower areas of reinforcement, but is unconservative otherwise. The method depends on the calculation of an enhancement to the small-deflection yield-line capacity which increases with increasing reinforcement size. Disproportionately higher slab capacities are obtained with higher reinforcement ratios, without adequate consideration of the capacity of the protected edge beams. The results show that the finite element analyses give a more logical indication of the contribution of the reinforcement area to slab panel capacity. The *Vulcan* 60-minute analyses show a steady increase in slab resistance with increasing reinforcement area, as they realistically consider the behaviour of edge beams and the failure properties of concrete and reinforcement.

3.3 Other Slab Panel failure mechanisms

In practice, slab panels are usually continuous over at least two supports. Continuity should provide higher slab panel resistance in fire. However, depending on the extent of the fire scenario in the building and the lightness of the reinforcement used in composite floor construction, this continuity could be lost, or significantly higher loads could be imposed on a protected perimeter beam between two adjacent slab panels. Coupled with thermal degradation of mechanical properties, these beams can experience fairly large deflections, and may collapse. Therefore it is prudent to examine the possible collapse mechanisms which could develop in these slab panels, so that they can be monitored in designs which employ the Bailey-BRE method or similar simplified methods, in order to ensure that each panel can develop its full tensile membrane capacity and not fail by the loss of the support from its protected beams. An examination of all possible scenarios offers the possibility of selecting the mechanism which requires the least plastic energy. A discussion of possible collapse scenarios is therefore provided in this section.

Figure 10 shows some potential slab panel folding failure mechanisms:

- The simple folding mechanism discussed earlier is shown as Mechanism 1. This applies to the isolated panel shown, which of course could represent panels whose rebar has fractured across their support beams.
- Collapse Mechanisms 2a and 2b can occur when multiple slab panels are exposed to a fully developed fire. These are essentially the same as Mechanism 1, but with additional hogging fold-lines along the assumed-undeflected support beams.
- Mechanism 3 is typical for an edge slab panel, in which the beam at the edge could be considerably lighter than those in the interior parts, and does not benefit from continuity. However this could have catenary support from adjacent floor panels.
- For corner slab panels, a mechanism of the general type shown as Collapse Mechanism 4 could be appropriate, although the exact location and alignment of the sagging fold line would depend on the least-energy mechanism.

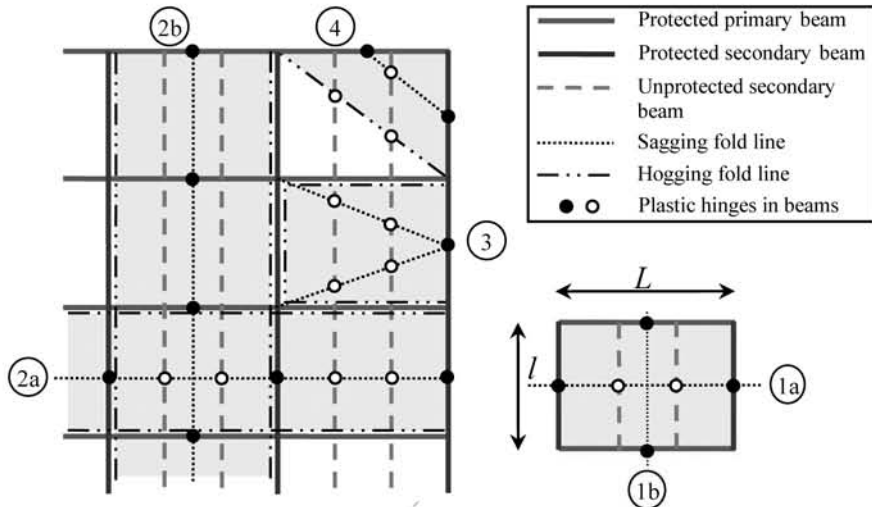


Figure 10: Potential Slab Panel Failure Mechanisms

Incorporating such folding mechanisms into the simplified approaches will help make them more robust for design by providing the resistance limits that these methods currently lack.

4 CONCLUSIONS

Both the original Bailey-BRE Method and TSLAB have based their acceptance criteria for fire resistance on the compartmentation integrity criterion, which is one of the three criteria which have to be satisfied, either in standard testing or in fire resistance design. This is almost certainly the key criterion when ideal conditions of vertical slab-panel support are maintained, and the deflection limits used appear justifiable for the prescribed mode of failure due to the occurrence of a single tension fracture across the slab's shorter span. However the results of these studies show that, since overall structural stability ("resistance" in Eurocode jargon), is not included in the method, its predictions can lose their conservatism with higher reinforcement ratios. The method's reliance on calculating the deflection required to enhance the traditional yield-line capacity without considering the stability of the edge beams results in very optimistic predictions of slab panel resistance with larger mesh sizes. The finite element analyses, on the other hand, show that, given the load redistribution which takes place, and the effects of

aspect ratios and edge beam deflections, only marginal increases in slab panel capacity are obtained with increasing reinforcement size, and the slab panel eventually fails by overall folding. Further analyses of the effect of reinforcement area on slab panel capacity has revealed that, for small-sized panels, and for lower fire resistance requirements, increases in reinforcement area does not significantly increase the capacity. Larger mesh sizes are required for large panels, and higher reinforcement ratios are also required for longer fire resistance times to resist the large initial thermal bending which takes place. In terms of membrane-action enhancement, however, there is little influence from increasing the mesh size. The simple edge beam collapse mechanism has been found to give accurate predictions of runaway failure of slab panels. Including this mechanism in the Bailey-BRE Method would therefore make its design predictions more realistic, at least for slabs of certain sizes and aspect ratios.

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