Title

“STRUCTURAL FIRE DESIGN OF UNPROTECTED STEEL BEAMS SUPPORTING COMPOSITE FLOOR SLABS ”[(1)]

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Abstract

It is common practice for all exposed structural members within a steel-framed building to have some form of applied fire protection to ensure that they retain their strength and stiffness during a fire. The use of applied fire protection is considered to be a distinct disadvantage for adopting a steel frame, compared to using other materials, due to the cost and time to apply the protection. This paper presents a new design method where 40 to 50% of the steel beams within a building can be left unprotected, provided a composite floor system is adopted. The design method utilises membrane action of the composite floor slab at large vertical displacements which are typically experienced during a fire. Careful specification of the location of the protected beams, within the floor plate, will allow membrane action to occur in the floor slab, allowing the static load to be redirected away from the unprotected steel beams towards the protected beams and columns.

Key-words:

Fire Design, Membrane Action, Unprotected Steel Beams, Composite Floor Slabs.

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1- INTRODUCTION

Due to its high thermal conductivity steel heats up very quickly when exposed to a fire and, similar to other materials, reduces in strength and stiffness at elevated temperature. For any building, the designer must ensure that in the event of a fire there are adequate means of escape for occupants, safe conditions for fire fighters, and the overall structural stability together with compartmentation is maintained.

For steel-framed buildings it has been typical practice for all exposed steel members to be insulated with some form of applied fire protection to ensure structural stability during a fire. The use and specification of the fire protection will ensure that the temperature of the supporting steel structure remains sufficiently low so that adequate strength and stiffness is maintained. Experience has shown that the use of applied fire protection generally results in safe buildings. However, the specification of the fire protection is based on tests carried out on isolated structural members (whether it is a beam, column, or section of floor slab) and ignores the true structural behaviour of the building in a fire.

Recent research (Bailey & Moore 2000, Huang *et al* 1999, Gillie *et al* 2002) has begun to investigate the structural response of entire steel-framed buildings in a fire, particularly buildings that incorporate composite floor slabs (comprising steel deck, normal or lightweight concrete and mesh reinforcement). This research has shown that the behaviour of single member tests, which current design methods are based on, does not resemble the behaviour of similar members when they form part of an entire building. Alternative load-path mechanisms from those commonly assumed for the ultimate and serviceability limit states have been identified within the structure during a fire which can be beneficial for the survival of the building. At the core of this research is the large compartment fire tests (Bailey *et al* 1999) that were conducted on the full-scale steel-framed building at the Building Research Establishment Laboratories in Cardington (Figure 1).

Figure 1 – Fire test on the Cardington steel building (Test 6)

By understanding the true behaviour of steel-framed buildings, when subjected to a fire, it is possible to identify, and apply fire protection, only to the steel members that are required to retain their strength and stiffness to ensure overall structural stability of the building. This approach represents a significant advance on the current philosophy of applying fire protection to all the exposed areas of the steel structure.

This paper discusses a design method that incorporates the true behaviour of an entire flooring system consisting of a composite floor slab and grillage of downstand steel beams. The method typically results in 40 to 50% of steel beams being left unprotected for a given fire resistance period, whereas current design methods typically result in all the steel beams requiring applied fire protection.

2- OVERVIEW OF FULL-SCALE FIRE TESTS

The steel-framed test-building at Cardington (Bailey *et al* 1999) was designed and constructed to resemble a typical modern UK city centre eight-storey office development. On plan, the building covers an area of 21m x 45m with an overall height of 33m. The structure was designed as a braced frame with lateral restraint provided by cross bracing around the three

vertical access shafts. The beams were designed as simply-supported acting compositely (via shear studs) with the supported floor slab. The composite floor slab was nominally 130mm deep and consisted of a steel trapezoidal deck with lightweight concrete and anti-crack mesh (6mm diameter bars at 200mm centres). The imposed load was achieved using sandbags, each weighing 11kN. This resulted in an overall applied load (dead plus imposed) of 4.9kN/m^2 per floor.
Six fire tests were conducted between 1995 and 1996, which ranged from heating one beam, to heating 2/5 of the floor plate. In all the tests the underside of the composite floor and the supporting steel beams were left unprotected. The columns, situated within the fire compartment, were protected to ensure vertical stability. No structural collapse was observed in any of the tests, with the most severe test, in terms of temperature, being Test 6 (Figure 1) where the atmosphere temperature exceeded 1200°C and the steel temperature of the exposed beams reached a maximum of 1150°C.
Current design methods (BS5950-8, ENV1994-1-2) estimate that structural collapse would occur when the beams reach 680°C, which is significantly lower than the temperatures recorded in the tests. The comparison between the tests and current design methods showed that the true behaviour of the building is not being considered in the design approach. In particular, the behaviour of the composite floor was shown to be predominately membrane action which provides a greater inherent fire resistance compared to the current design assumption which is limited to flexural action. By considering membrane action of the composite floor system, in the fire design, it is possible to safely specify steel-framed structures where only a proportion of the steel beams, within the floor plate, require applied fire protection.

3- MEMBRANE ACTION IN FLOOR SLABS

Membrane action of concrete floor slabs is due to the development of in-plane forces within the depth of the slab. Depending on the horizontal restraint conditions around the slab's perimeter membrane action can occur at small and large vertical displacements. During a fire large displacements of floor slabs within a building are acceptable provided structural collapse or compartmentation failure does not occur. These large displacements lead to membrane action occurring within the slab, which can be beneficial to the survival of the building.

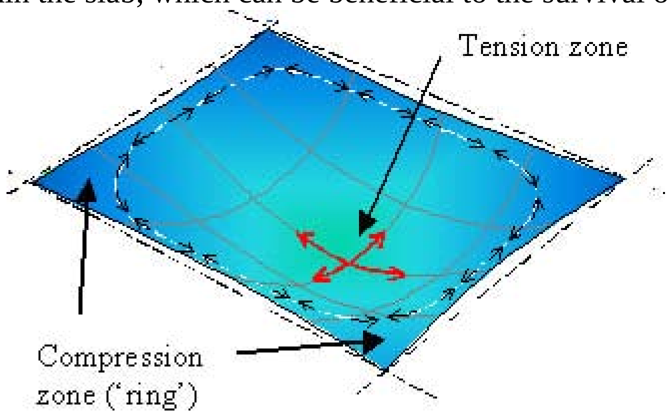

Figure 2 - Membrane action of a floor slab with no horizontal restraint around its perimeter.

For concrete slabs, which are horizontally restrained around their perimeter and subjected to large vertical displacements, membrane action will develop with the reinforcement utilising its full tensile capacity and supporting the load by acting as a kind of net. This type of behaviour is commonly referred to as tensile membrane action. For slabs that have no horizontal restraint around their perimeter, membrane action can still develop provided the slab is two-way spanning. The slab supports the load by tensile membrane action occurring in the centre of the slab and compressive membrane action forming a supporting 'ring' around the perimeter of the slab (Figure 2). This behaviour is analogous to a bicycle wheel: the spokes representing tensile membrane action and the wheel rim representing compressive membrane action. Tensile membrane action, which develops at large displacements, is dependent on geometry with increasing vertical displacements resulting in an increase in load-carrying capacity.

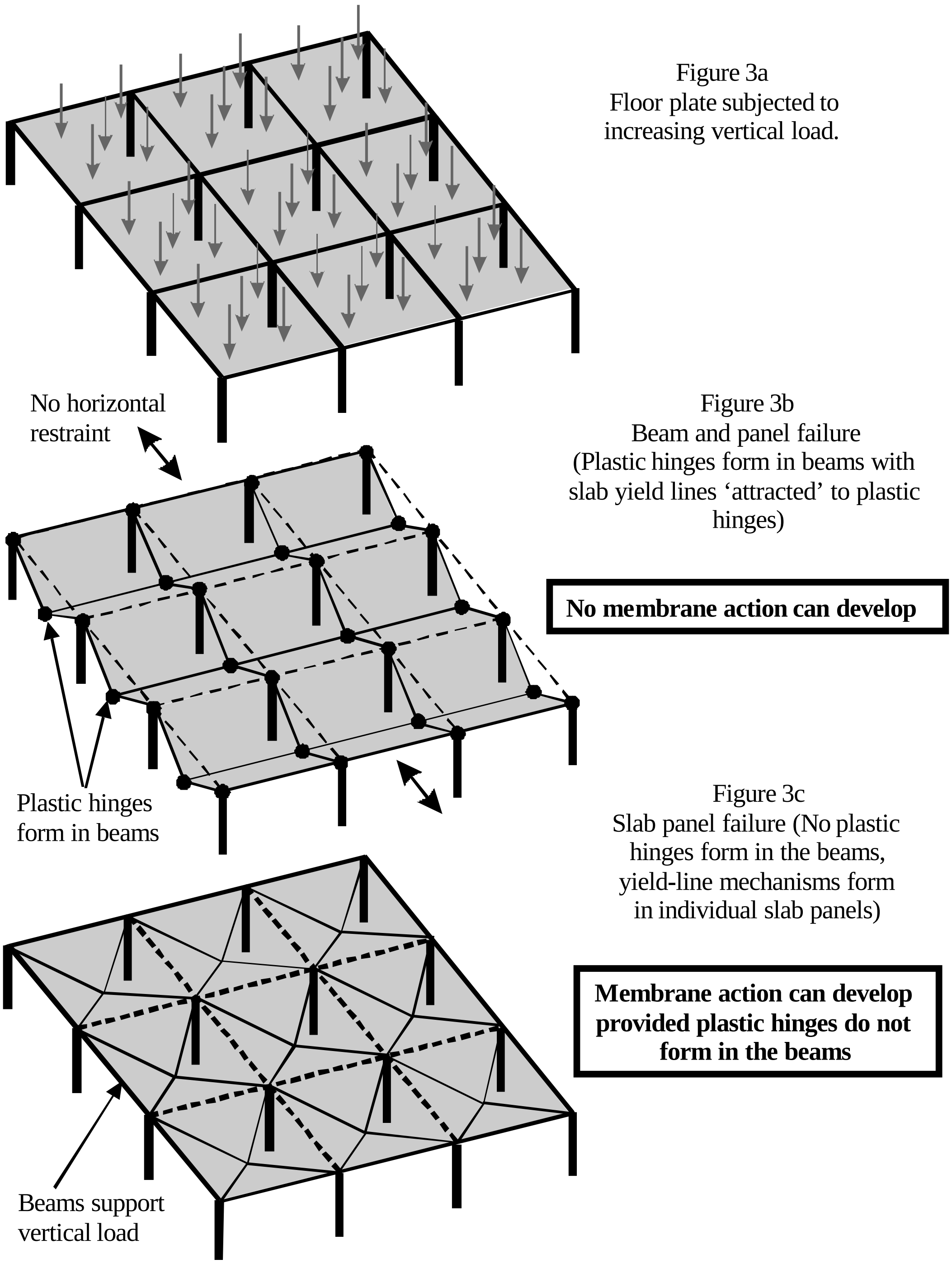

Figure 3 - Flexural failure of floor plate under increasing vertical load

3.1- Design Requirements For Membrane Action

Tensile membrane action in floor slabs will develop at large displacements if there is horizontal restraint to the floor slab, or the slab is two-way spanning allowing a perimeter compressive restraining ring to form within the depth of the slab. For example, if the floor plate shown in Figure 3a, which consists of a slab supported by a grillage of beams and columns, is loaded with a continually increasing uniform vertical load, two modes of flexural failure are possible. If the design of the beams is such that plastic hinges form, as shown in Figure 3b, a mechanism consisting of yield-lines across the whole floor plate occurs with the floor plate effectively folding along the yield-lines. Membrane action cannot occur in this mechanism since restraint against lateral movement is not available. However, if the beams are designed such that no plastic hinges form, the ultimate flexural resistance will be governed by the individual slab panel behaviour (Figure 3c). For this flexural mechanism, provided that each panel is vertically supported around its perimeter, membrane action can occur in the slab irrespective of whether horizontal restraint around the edge of each panel is provided or not. If membrane action is included in the design, the supporting beams around the perimeter of the slabs must be able to support the vertical load. If plastic hinges form after membrane action develops in the slab the mechanism shown in Figure 3c will change to the mechanism shown in Figure 3b with structural collapse occurring.

3.2- Design Requirements For Membrane Action To Resist Fire.

The same principle used to ensure membrane action develops within a floor plate under increasing vertical load can be used for the fire design of composite floor slabs supported by steel beams. In a fire the vertical load remains constant and the resistance of the floor slab reduces, as it is heated.

To allow membrane action to develop in the composite floor slab a specific number of steel beams are protected, to ensure that they retain their strength during the fire, with the protected beams forming the perimeter of a number of composite slab panels (Figure 4). Since the protected beams provide vertical support to the bounded slab panels, during a fire, membrane action will develop within each panel. The panels can only be square or rectangular and the steel beams within each panel are left unprotected. In a fire, the load is supported by the composite slab, using membrane action, with the load being transferred to the protected beams and then to the columns, which are also protected.

With the protected beams dividing the floor plate into a number of slab panels, the question arises as to whether these panels, during a fire, should be considered as unrestrained or restrained against horizontal movement. Horizontal restraint is beneficial to the ultimate strength of the slab since it can allow the full tensile capacity of the reinforcement to be utilised and thus increase the load-carrying capacity of the slab. However, if the edge of the slab panel coincides with the edge of a building or a service duct then it is horizontally unrestrained along that edge. If the slab panel is located in the centre of the floor plate, it could be argued that the panels are restrained horizontally since the reinforcement is continuous over the protected beams that form its perimeter. However, large hogging moments will occur over the vertical support provided by the protected beams which, together with the membrane forces in the floor slab, will generally lead to fracture of the reinforcement in this area. Therefore, for design purposes, a conservative assumption of each slab panel having no horizontal restraint (i.e. the reinforcement fractures in the hogging regions) is assumed. This conservative assumption also simplifies the design method since all slab panels are considered to be unrestrained irrespective of their location within the floor plate.

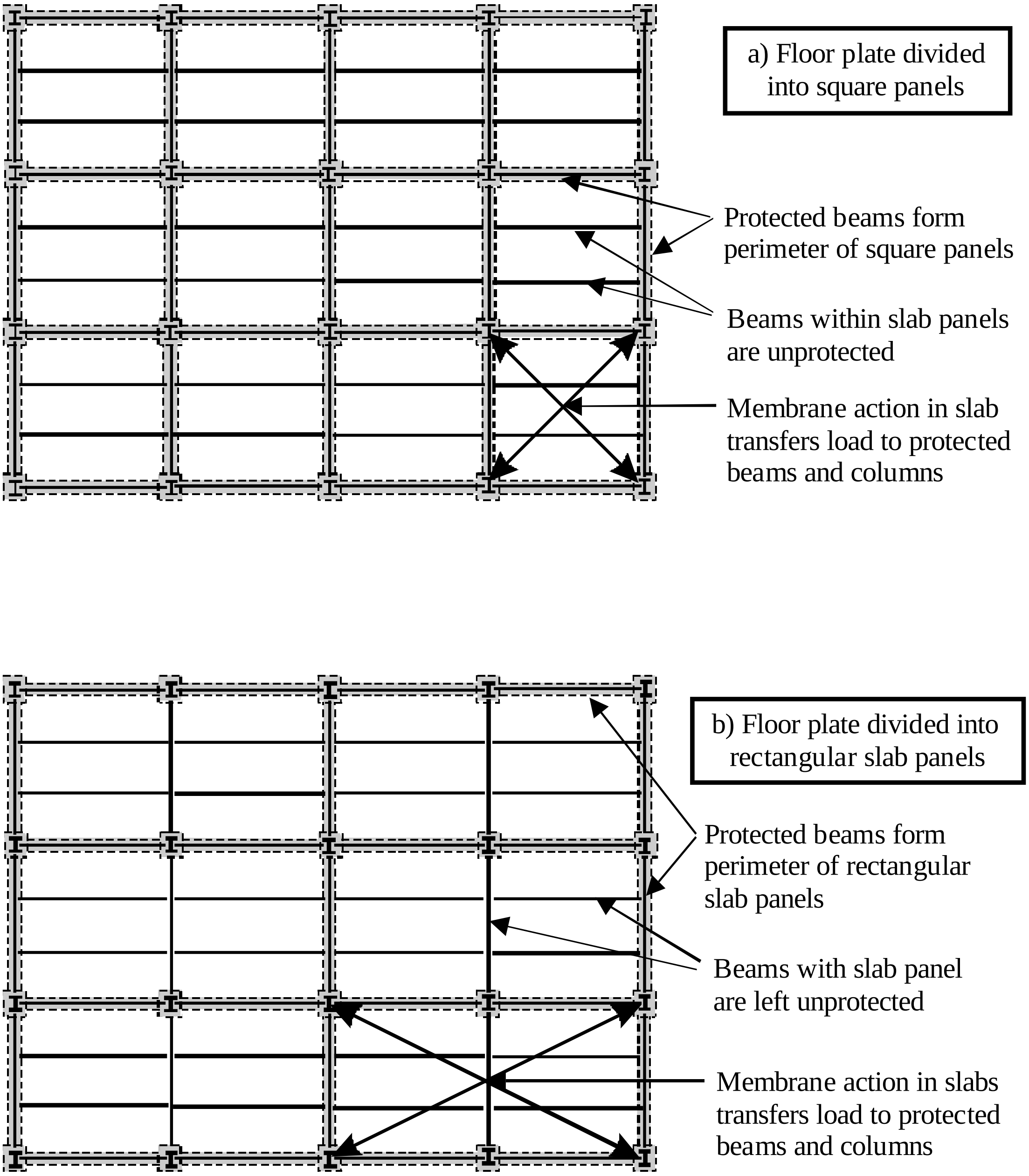

Figure 4 - Two options for dividing the floor plate into slab panels to ensure membrane action in the floor slab during a fire.

4.0- FIRE DESIGN OF COMPOSITE FLOOR SLABS

Once the floor plate is divided into a number of slab panels the load capacity at the fire limit state of each panel ($w_{p\theta}$), consisting of the composite floor and unprotected steel beams, is given by the energy equation,

$$w_{p\,\theta} = e\left(\frac{Internal\ work\ done\ by\ the\ composite\ slab\ in\ bending}{External\ work\ done\ by\ the\ floor\ system\ per\ unit\ load}\right) + \left(\frac{Internal\ work\ done\ by\ the\ beam(s)\ in\ bending}{External\ work\ done\ by\ the\ floor\ system\ per\ unit\ load}\right) \quad (1)$$

where,
e = an enhancement due to membrane action in the slab.

For simplicity, catenary action of the unprotected beams is ignored in Equation (1). The energy equation is based on the yield-line pattern of the slab which is continually changing, as the slab is heated, due to the continual reduction in strength of the unprotected steel beams. The change in the pattern of the yield-line, for a given slab, is best shown by a simple example.

Consider a 9.0m by 6.0m heated area of slab with one unprotected secondary beam (Figure 5). The mode of behaviour of the system changes with increasing temperature, as shown in Figure 5. The change from one mode to the next is dependent on the strength of the steel composite beam, which is reducing as it is heated in the fire. The final mode of behaviour, as the capacity of the composite beam tends towards zero, is due to the 9.0m by 6.0m simply supported, two-way spanning, slab supporting the entire applied load.

For all the yield-line patterns, shown in Figure 5, membrane action will develop in the slab with the generated in-plane forces being greater for a 'fan' type of pattern compared to the 'back-of-an-envelope' pattern. It has been shown from ambient temperature tests, on unrestrained slabs, that once membrane action develops in the slab the shape of the yield-line pattern does not change with increasing vertical displacement. However, this is not true in a fire situation, since the supporting steel beam, which controls the shape of the yield-line pattern is continually reducing in strength. This will result in a change in the mode of behaviour with increasing temperature (Figure 5). With continual changes in the mode of behaviour and continual changes in membrane action (which is dependent on the mode of behaviour), it can be seen that applying Equation (1), even to the simple example described above, can be complicated, time consuming, and unsuitable as a hand calculation design method.

4.1- Design Assumptions

By assuming that the dominant load-carrying capacity of the system is due to the composite slab, the following assumptions can be applied to obtain a conservative estimate.

1. The load supported due to the flexural behaviour of the composite slab is calculated based on the lower-bound yield-line mechanism, assuming that the beams have zero resistance.
2. The enhancement due to membrane action in the composite slab (*e*) is based on the lower bound yield-line mechanism of the slab.
3. The load carried by the flexural behaviour of the grillage of composite beams is calculated assuming that the beams are simply-supported and support a loaded area calculated assuming the slab is simply supported (i.e. typical assumptions taken for normal design).
4. The load-carrying capacity of the composite beams and slab (enhanced due to membrane action) are added together, as shown in Equation (1).

The above assumptions result in a conservative estimate of the load-carrying capacity and a useable design method. Equation (1) can now be re-written in a simplified form as:

$$w_{p\theta} = e\,(\mathit{yield\text{–}line\ load\ of\ slab}) + (\mathit{load\ carried\ by\ unprotected\ beams}) \qquad (2)$$

Some small errors are introduced by adopting this simple representation of the true energy equation, but these errors are nominal and have been discussed in detail previously elsewhere (Bailey & Moore 2000).

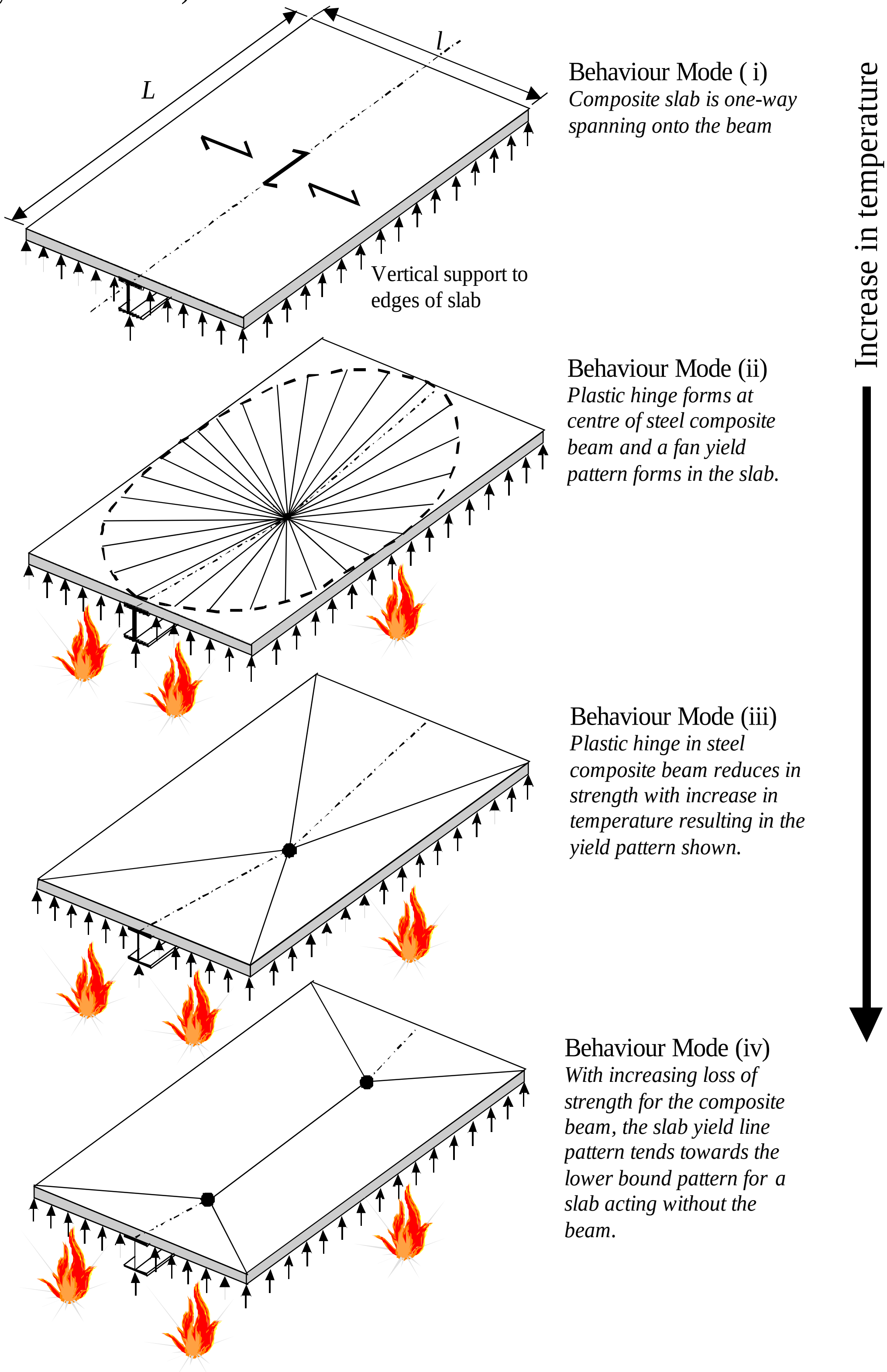

Figure 5 – Yield line pattern for slab with heated supporting steel beam

4.2- Calculation of load-carrying capacity

Considering Equation 2, the designer needs to calculate the load-carrying capacity of the grillage of beams, the yield-line load of the slab panel and the enhancement factor due to membrane action, based on a maximum allowable vertical displacement. Determination of the terms in Equation 2 is discussed below.

4.2.1- Calculation of load carried by the unprotected composite beams in fire.

The load carried by a grillage of beams is based on the load-carrying capacity of the beam, within the system, which will fail first. The resistance of the beam that fails first is converted into a load-carrying capacity, in terms of kN/mm^2 by considering the total area that it supports, assuming that the beams and slab are simply supported.

The moment resistance of the composite beam, at a given temperature, is given by,

$$M_{\theta} = R\, M_{c20°C} \tag{3}$$

where,

M_{θ} is the moment capacity of the composite beam at the fire limit-state

R reduction factor shown in Table 4 based on the steel temperature of the bottom flange.

$M_{c20°C}$ is the moment capacity of the composite beam at ambient temperature.

To calculate the reduction factor (R), the temperature of the bottom flange of the steel beam is required for a given fire resistance period. This can be obtained from BS5950-8 (Figure 6), based on the flange thickness or by using the calculation method given in ENV1994-1-2. Once the temperature is obtained, the reduction factor is calculated using Table 1. The moment capacity can be calculated using Equation 3 and the load carried by the grillage of beams estimated.

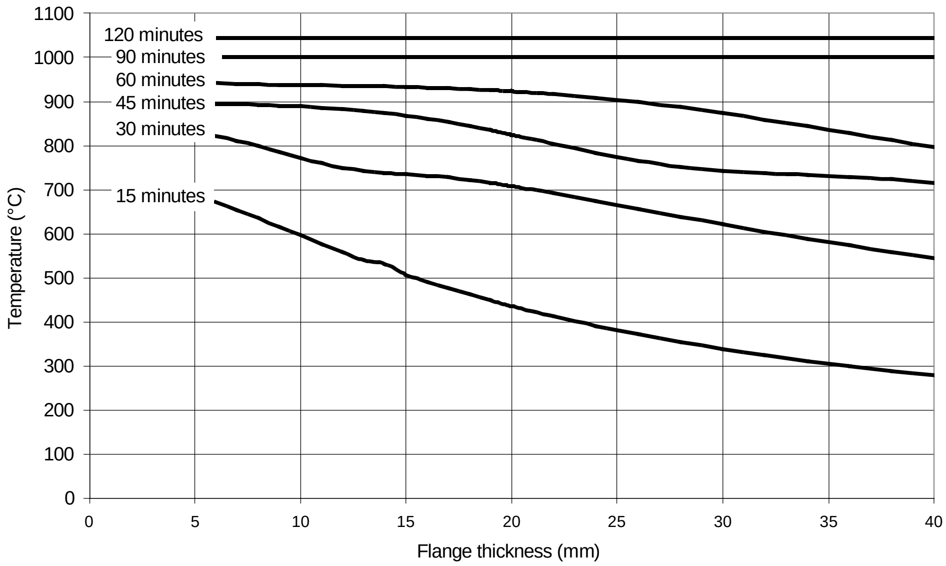

Figure 6 Temperature of bottom flange of a steel composite beam for a given fire resistance period.

Temperature of bottom flange (°C)	526	558	590	629	671	725	820	860	900	1000	1100	1150
Reduction Factor (*R*)	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.08	0.06	0.04	0.02	0.01

Notes

1. Interpolation can be used

Table 1 *Reduction factors for unprotected steel composite beams supporting a floor slab.*

4.2.2- Calculation of the yield-line load

For rectangular or square simply-supported slabs the moment capacity can be calculated using (Wood 1961),

$$m = \frac{pl^2}{24}\left[\sqrt{3+\left(\frac{l}{L}\right)^2} - \frac{l}{L}\right]^2 \tag{4}$$

where,

m is the sagging moment capacity of the composite slab at the fire-limit state.
p is the applied load at the fire-limit state.
l is the shorter span of the slab panel.
L is the longer span of the slab panel.

Using Equation (4) the value of pl^2/m can be calculated for various aspect ratios (Table 2), leading to an assessment of the load-carrying capacity (p). The moment capacity of the slab (m) is dependent on the position of the reinforcement, the strength of the materials used and the temperature through the cross-section. The calculation is simple and is shown in current design codes (BS5950-8, ENV1994-1-2) and design guides (Newman 1991).

$\frac{L}{l}=$	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	5.0	10	∞
$\frac{pl^2}{m}=$	24	20.3	17.9	16.2	15.0	14.14	12.6	11.73	10.1	9.0	8

Table 2 *Calculation of yield-line load for slabs with isotropic reinforcement.*

4.2.3- Calculation of enhancement due to membrane action

The in-plane stresses are estimated, for a horizontally unrestrained slab, based on the observed behaviour of a number of tests at ambient temperature. The derivation of the enhancement due to membrane action is given elsewhere (Bailey 2001) and can be represented as design charts, as shown in Figure 7. The use of the design charts depends on the amount of compressive force present during flexural action of the slab (i.e. the proportions of the stress block under flexural action) and is defined by the variable g_0 for each design chart. Interpolation between the charts for different values of g_o is permitted.
Each chart specifies the enhancement, above the yield-line load, due to membrane action, based on the aspect ratio (a) of the slab and the allowable vertical displacement of the slab. The charts represent the displacement as a ratio (displacement/effective depth) where the effective depth can be taken as the average depth of the reinforcement from the top of the slab.

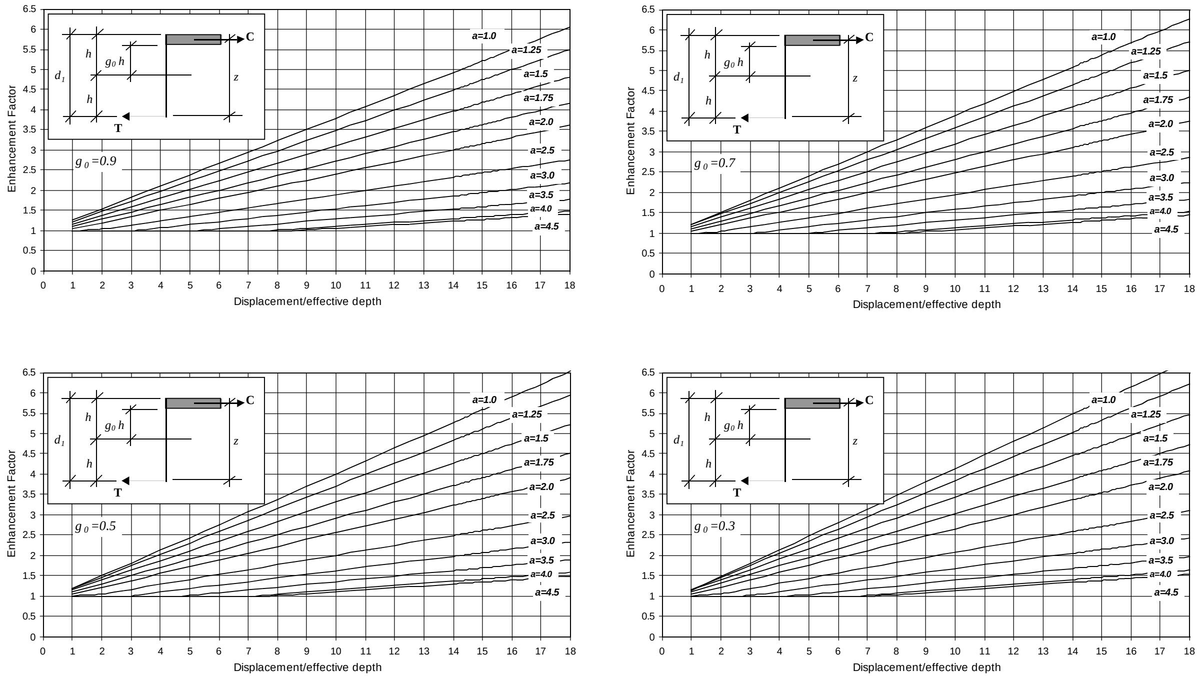

Figure 7 - Enhancement factors for isotropic reinforced slabs

The enhancement factor is based on the maximum allowable vertical deflection. Observations from tests have shown that failure of horizontally unrestrained concrete slabs is due to fracture of the reinforcement in the longer span. Therefore, to predict failure, the strains in the reinforcement need to be estimated. This is an extremely difficult task at elevated temperature, where the mechanical and thermal effects need to be considered. A pragmatic approach was adopted (Bailey & Moore 2000), to derive an equation defining the maximum displacement, which was correlated against all available test data at ambient and elevated temperatures. The maximum allowable displacement is given by,

$$v = \frac{\alpha\left(T_2 - T_1\right)l^2}{19.2h} \quad + \quad \sqrt{\left(\frac{0.5\,f_y}{E}\right)_{Reinf't_{20°C}} \frac{3L^2}{8}}$$

but, (5)

$$v \quad < \quad \frac{\alpha\left(T_2 - T_1\right)l^2}{19.2h} \quad + \quad l\,/\,30$$

where,

α	is the coefficient of thermal expansion of concrete (18×10^{-6} for normal weight concrete and 8×10^{-6} for lightweight concrete).
$(T_2 - T_1)$	is the difference between the temperature at the top and bottom of the slab. For design purposes the value of $(T_2 - T_1)$ for composite slabs can be taken as 770°C for up to 90 mins fire resistance and 900°C for 2 hours fire resistance.
l	is the shorter span of the slab panel.
L	is the longer span of the slab panel.
f_y	is the yield stress of the reinforcement at ambient temperature.
E	is the Youngs Modulus of the reinforcement (=210000 N/mm^2)

h is the thickness of the slab (For trapezoidal slabs take depth to mid-height of troughs and for dovetail slabs assume full depth of slab).

4.3- Calculation of load on protected beams

By using membrane action of the composite floor slab to effectively span over the unprotected beams and between the protected beams, the load on the protected beams will be different compared to that calculated using the assumptions adopted in normal (cold) design. For example, if the structural grid layout in Figure 8 is considered, the load on the protected beams could conservatively be assumed to consist of the triangular and trapezoidal loaded areas of the supported slab, defined by the yield-line pattern. This will result in higher loads on beams (1) and lower loads on beams (2), compared with the assumptions of loaded tributary areas typically adopted for normal (cold) design. If the steel beams (1) are designed assuming a trapezoidal load it is assumed that the beams will remain infinitely stiff compared to the slab. Considering flexural action, as the stiffness of the beams decreases, the load will tend towards the corners of the slab. If the mechanics of membrane action are considered the load will also tend towards the corners of the slab (Figure 9), suggesting that the load distribution on the protected beams shown in Figure 8, which is defined by the yield line load, is conservative. Lack of experimental data is such that no definitive guidance can be given with confidence. Therefore for current design, it is recommended that the conservative load distribution shown in Figure 8 be adopted.

Once the load on the protected beams is calculated the required capacity/protection to these beams should be determined using BS5950-8 or ENV1994-1-2.

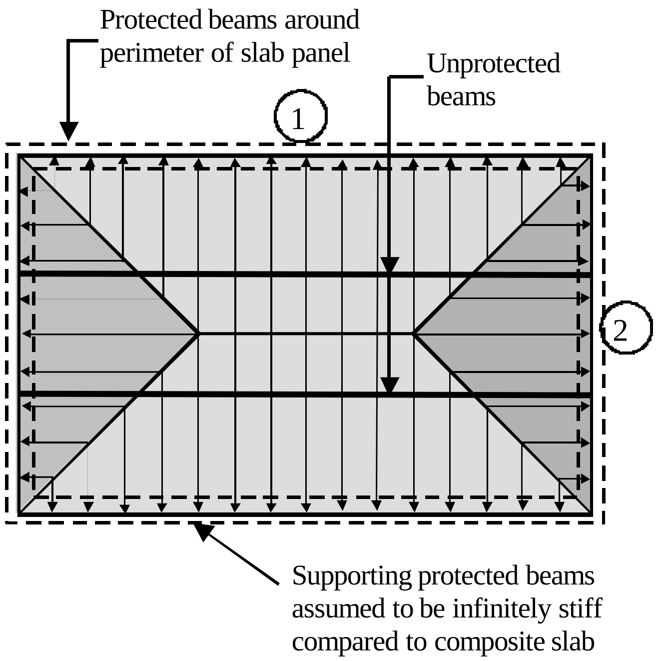

Figure 8 *Assumed load distribution onto infinitely stiff supporting beams*

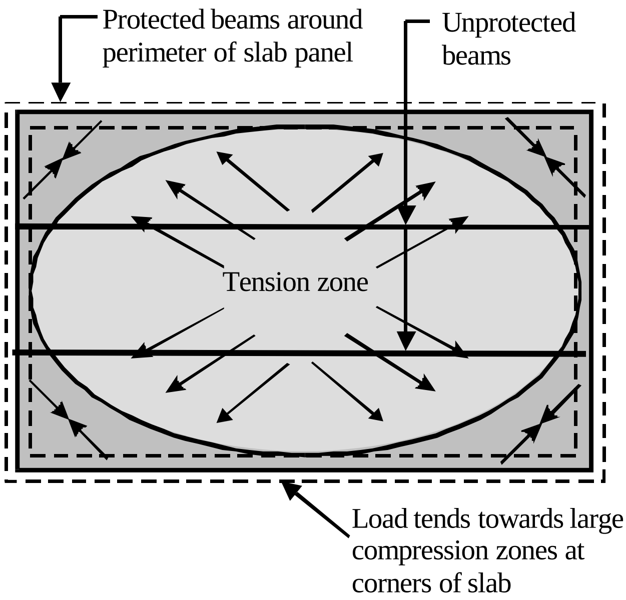

Figure 9 *Postulated load distribution onto supporting beams due to membrane action in the slab*

5.0- DESIGN EXAMPLE

Consider the floor plate shown in Figure 4 where beams are protected such that 9m by 9m slab panels are formed and the steel beams within each panel are unprotected. The composite secondary and primary beams are assumed to be 356×127×39UB and 533×210×92UB respectively. The fire resistance period is assumed to be 60 mins and using BS5950 Part 8 or Figure 6 the temperature of the unprotected steel beams are estimated to reach a maximum temperature of 938°C. At this temperature the unprotected beams can support a load of 0.68kN/m^2. Since the applied load on the structure for a typical office building at the fire limit-state is approximately 6.0kN/m^2, it can be seen that the load needs to be predominately carried by membrane action of the composite slab. This allows the applied load to be redistributed away from the unprotected beams and onto the protected beams around each panel. If membrane action is ignored, as adopted by current design methods, all the steel beams will require protection to ensure that they retain enough strength for the duration of the fire.

Assuming a standard A252 mesh reinforcement (8mm diameter bars at 200mm centres) the moment capacity of the slab can be calculated using design codes (BS5950-8, ENV1994-1-2) or guides (Newman 1991), ignoring any contribution from the steel deck. If the mid-depth of the mesh is placed 35mm above the deck, the moment capacity of the slab is 4.3kNm, which using a standard yield-line analysis gives a load-carrying capacity of 1.3kN/m^2.

The enhancement factor is based on the maximum allowable displacement given by Equation 5. Assuming,

α =	8×10^{-6} (lightweight concrete, EC4 Part 1.2)
T_2-T_1 =	770°C (from test results for 60 mins fire resistance)
f_y =	460N/mm^2 (current UK characteristic value for reinforcement)
E =	210000N/mm^2
h =	103mm (depth to the mid-height of the troughs of the slab).

The maximum allowable vertical displacement, given by Equation 5, is 435mm. With an effective depth of 40mm, and using Figure 4, an enhancement factor of 4.1 is obtained. Therefore, the load-carrying capacity of the floor slab, considering a 9m by 9m slab panel, is given by $(4.1\times1.30) + 0.68 = 6.0\text{kN/m}^2$, which equals the applied load. Therefore the beams within the 9m by 9m slab panels can be left unprotected.
The protected secondary beams around each panel will carry a higher load at the fire limit-state, due to membrane action in the slab, compared to that assumed using the typical design assumptions at the ultimate limit-state. These beams need to be checked for this increase in load.
Figure 10 shows an office building constructed in Hatfield in the UK, which was designed to achieve 60 minutes fire resistance. By using the design method discussed in this paper a significant number (45%) of the steel beams were left unprotected.

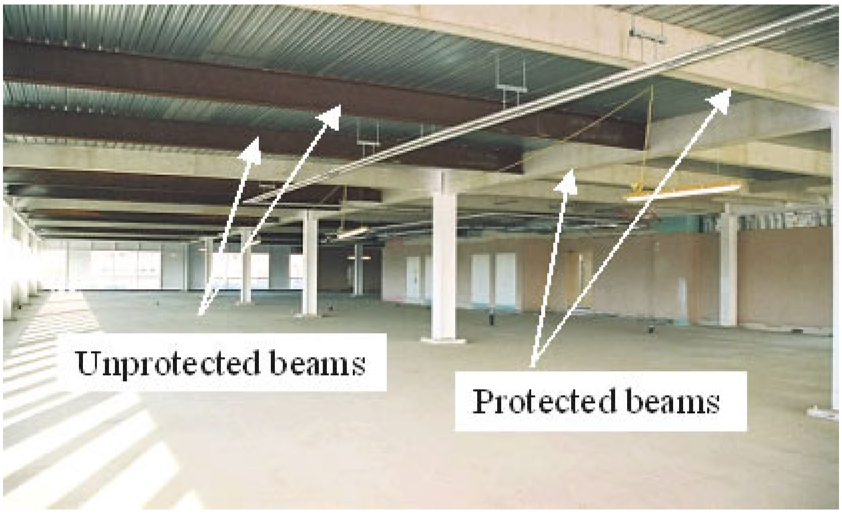

Figure 10 Example showing unprotected and protected beams by using membrane action in the composite floor slab.

6.0 – CONCLUSION

Based on the observations and results from a series of large compartment fire tests, on a full-scale steel-framed building, a simple new fire design method has been presented for steel-framed buildings that incorporate downstand beams and a supporting composite floor slab. The design method, for the first time, includes the membrane action in the composite floor slab, which occurs at large displacements. By including membrane action a significant number (approximately 40-50%) of the steel downstand beams can be left unprotected, resulting in cost savings in terms of material used and a reduction in the required time to fix the fire protection.
The design method involves dividing the floor plate into a number of slab panels, where protected beams form the perimeter of the panels and the beams within the panels are left unprotected. Membrane action allows the composite floor slab to bridge over the unprotected beams, transferring the load to the protected beams and then to the protected columns. The size of the panels and thus the number of unprotected beams can be governed by the designer, since the membrane action of the composite slab can be enhanced by adding more reinforcement within the depth of the concrete.

There are a number of conservative assumptions embodied within the design method. These assumptions include ignoring the continuity of the slab over the protected beams, ignoring the contribution from the steel deck, ignoring catenary action in the steel beams and defining the yield-line pattern of the slab assuming zero resistance from the unprotected beams. Although it is accepted that these assumptions are conservative, and necessary to present a usable design approach, more research work is required to define the extent of the conservatism.
The failure criterion within the design method is based on an estimate of the maximum vertical displacement where fracture of the reinforcement will occur. Estimating the strains in the reinforcement, to define the point at which fracture occurs, is extremely difficult at elevated temperatures where both mechanical and thermal strains need to be considered. A pragmatic approach was presented defining the displacement at which fracture would occur. Comparisons with all available test data show that the estimate of the maximum displacement is conservative. However, more research work is required to refine the maximum allowable displacement and the failure criterion (or criteria) in general.

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