Behavior of Structures in Fire and Real Design – A Case Study

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ABSTRACT: A great deal of work on the behavior of composite steel–concrete structures in fire has been developed since the Cardington frame fire tests (UK) conducted in the 1990s. This has now been broadened so that the design of structures to resist fire has a real engineering basis and is not reliant on results from single element testing in the standard furnace.

Several projects involving office buildings in the UK and abroad have highlighted the need for developing the understanding of whole frame behavior in fire. Since the collapse of the World Trade Center in New York City in 2001 (9/11), robust engineering solutions incorporating the response of a building to fire are in great demand. The basics of structural mechanics at high temperatures can be used in such designs to understand the fire response of many structures with the aid of computer modeling.

This article provides a direct comparison between the structural response of an eleven-story office building in the city of London, when designed in a prescriptive manner with applied fire protection on all load bearing steelwork, and the response of the same structure designed using a performance-based approach leaving the majority of secondary steelwork unprotected. The intent is to demonstrate that structural stability during a fire can be maintained in specific cases without relying on passive fire protection.

This study contributes to the field of structural fire engineering by extending the research work previously conducted by the author to a real design case and addresses the issues raised by approving authorities, insurers, and the client when a fire engineered approach is used to calculate structural response to fire. It also demonstrates the use of advanced analysis to understand beam-core connection

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response in a fire, as part of a series of global finite element analyses to ensure that the unprotected structure proposed provides structural stability and maintains compartmentation for the design fires agreed upon among the necessary stakeholders in this project.

KEY WORDS: fire engineering, design fires, structural response, thermal expansion, performance-based design, prescriptive design, approvals process.

INTRODUCTION

RECENT RESEARCH IN the field of structures in fire has been used to provide a robust design solution to the passive fire protection arrangement for an eleven-story office building in London, UK. Detailed finite element analysis (FEA) allows engineers to examine the structural behavior of a composite steel frame as it continues to support loading during a fire. In some cases, this type of analysis permits a reduction in the number of steel beams that require passive fire protection, while maintaining structural stability and compartmentation. This form of analysis also highlights areas where the structure is less robust during a fire and where additional fire protection or structural measures may therefore need to be introduced. Robust structures at ambient temperatures may not necessarily be robust when exposed to fire. Especially since 9/11, the owners and occupiers of tall buildings have demanded that design teams predict the response of structures to fire as an essential part of the design and approvals process.

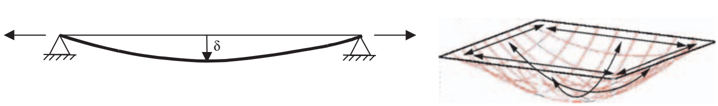
The Cardington frame fire tests [2] in the UK in the 1990s provided a wealth of experimental evidence about how whole frame composite steel–concrete structures behave in fire. The Cardington frame continued to carry the load during a number of full-scale fire tests despite, in most cases, having no fire protection on any of the steel beams (unprotected steel often reached temperatures in excess of 900°C). The columns were generally protected to their full height. In all tests, there was considerable deflection of the composite floor slab in the region of the fire. However, the local and global stability of the structure was maintained and no breach of compartmentation was observed, floor-to-floor or floor-to-core.

Historically, fire resistance design of structures has been based upon single element behavior in the standard fire resistance test [3]. Engineers have always recognized that whole frame structural behavior in fire cannot be described by a test on a single element because it does not represent the alternative load paths in a full structural frame. However, it is only in relatively recent years since the Broadgate Phase 8 fire in London, UK [4] and the subsequent Cardington frame fire tests [2] that researchers have fully investigated and understood the behavior of whole frame composite steel–concrete structures subjected to fire.

The main conclusions from the tests and the subsequent research projects [2,5–8] were that composite framed structures possess reserves of strength by exhibiting large displacement configurations with catenary action in beams and tensile membrane behavior in the slab [5–8] (see schematic representation in Figure 1). The tensile membrane action in the slab is supported by a compression ring around the boundary of the slab and by anchoring of the reinforcement through the shear studs connecting the boundary steel beams to the slab. Furthermore, for the duration of the Cardington tests, thermal expansion and thermal bowing of the structural elements, rather than material degradation or gravity loading, governed the response to fire [5]. Large deflections were not a sign of instability, and local buckling of beams helped thermal strains to move directly into deflections rather than cause high stress states in the steel. Runaway failure (a rapid increase in the rate of deflection) was not observed in the Cardington tests. However, had failure occurred, researchers believe that gravity loads and strength would have been the critical factors near impending failure [5].

An indeterminate structure, such as a multistory frame, is capable of transferring load through many alternate load paths. This is true at ambient and high temperatures that occur in fire. Consequently, the pattern of forces and stresses in an indeterminate beam (as part of a structure) are determined by the relative stiffness of the other parts of the structure as well as equilibrium considerations. Compatibility of deflections in both directions or spans of a floor plate will also play a key role. If a structure has adequate ductility and stability, the redundancy under fire conditions enables the structure to find alternative load paths and mechanisms to continue supporting additional load when its strength has been exceeded at a single location.

In general, the key aim of the design and detailed analysis presented here was to meet the functional requirements of the Approved Document B (fire safety [9]) of the Building Regulations in the UK. In this context, this means that the compartmentation arrangements and passive fire protection be designed to maintain the stability of the structure for a



(a) 1D Catenary action in beams (δ = deflection)   (b) 2D Tensile membrane action in slabs

Figure 1. *Catenary action in beams and tensile membrane action in slabs. (The color version of this figure is available on-line.)*

reasonable period and limit fire and smoke spread to the floors above the fire floor.

FIRE RESISTANCE TESTING VERSUS WHOLE FRAME BEHAVIOR

The fire resistance levels recommended in regulatory documents are based on the behavior of single structural elements heated in a furnace with a standard fire exposure. The fire resistance of the structural element is taken as the time, to the nearest minute, between commencement of heating and the time at which failure occurs. Periods of fire resistance are normally specified as 0.5, 1, 1.5, 2, 3, and 4 h, respectively.

This measurement of fire resistance is known as the standard furnace test or the fire resistance test [3]. The test determines the ability of a building element to continue to perform its function for a period of time without exceeding defined limits. Specifically, for load bearing elements and/or separating elements of construction in the UK, BS 476 Part 20 [3] defines three criteria for insulation, integrity, and stability that must be passed in order to achieve a fire resistance rating. For stability of horizontal load bearing elements of structure, for example beam and floor slab, failure is defined at a deflection of $L/20$, or when the deflection exceeds $L/30$, failure can be defined as a rate of deflection of $L^2/9000d$ where, L is the clear span of the specimen under test and d is the distance from the top of the structural section to the bottom of the design tension zone. This limit is based on the size of a typical standard test furnace and the maximum deflection that can be achieved without causing damage to the furnace. Therefore, in a code compliant building in the UK, with all structural elements protected, a floor may deflect up to $L/20$. For a 7.5-m-long beam, this equates to 375 mm and for an 18-m-long beam, this is 900 mm.

Owing to its simplicity, the furnace test does not consider vital structural phenomena found in the 3D behavior of real buildings including:

- Large deflections.
- Restrained thermal expansion and thermal bowing.
- Membrane and catenary load carrying mechanisms in slabs and beams, respectively.
- Compatibility of deflections in two or more directions in an integrated structural frame, for example, in a long fire compartment, the long span will tend to deflect to a greater extent than the shorter perpendicular span because of the difference in length and the resultant thermal expansion in each direction.

THE BUILDING

The case study presented here to compare and contrast performance-based design and a 'code compliant' structural design for fire, consists of an eleven-story office building, eight storys above the ground and three below, at Mincing Lane in London.

The floor plate measures 40 m × 60 m (see Figure 2). There is a concrete core at the center of the building containing services and escape stairs, which are also designed for fire fighting. The cores are enclosed in 2-h fire rated construction and provided with dedicated fire fighter lifts, a rising main, and a lobby/vestibule separating the stairs from the accommodation. The floor slabs are compartment floors of composite steel and normal weight concrete construction. Composite action is achieved by shear studs between the top flange of the beams and the concrete dovetail deck slab. Over two-thirds of the floor plate, secondary, and primary steel beams span 9 m between the core and the main column line and then shorter beams (≈2.5 m long) span between the main column line and the masonry façade. To the rear of the building, primary and secondary steel beams span 10 m between the core and the column line on the façade.

Two sides of the building have a load-bearing stonework façade, which behaves as columns at 3-m centers. The remaining two sides are steel frames with cladding.

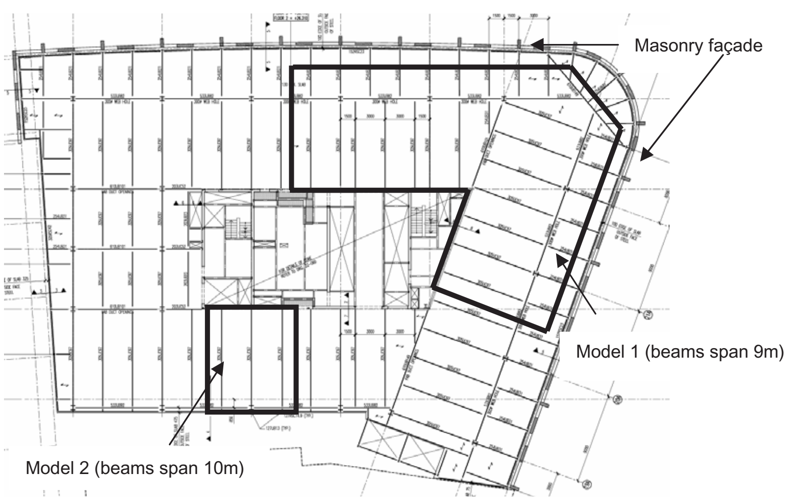


Figure 2. *Plan of the office building showing the extent of the global FE models.*

FAILURE CRITERIA IN COMPOSITE FRAME STRUCTURES

To assess the results provided from a FEA, some means of defining 'failure' must be established. Currently in the UK, there is no regulatory definition of 'failure'.

The term 'failure' is not straightforward to define in the context of this type of analysis. A compartment fire may lead to large deflections of main and secondary beams but it is unlikely to cause structural collapse, i.e., stability requirements can be met. However, large deflections could cause a breech of the separating function of a compartment floor or compartment wall, for example, the wall of the escape cores.

On this basis, the following aims and assessment methods were proposed to the stakeholders (client, insurer, fire authority, and building authority):

- Stability of the structure would be maintained throughout the design fire. This was primarily assessed by looking at the rate of deflections during the fire. Runaway deflections (a rapid increase in the rate of deflection) were assumed to indicate incipient failure of the floor system and pulling in of the columns.
- Horizontal compartmentation would be maintained for the duration of the design fire. This was also assessed by monitoring the rate of deflection of the composite floor. A rapid increase in deflection in any region of the floor plate was assumed to imply compartmentation failure.
- Vertical compartmentation via the vertical fire fighting shafts would be maintained for the duration of the design fire. This was assessed by monitoring the connections at the shaft wall to ensure that they maintained their capacity for the fire period.

THE GLOBAL FE MODELS

Two finite element (FE) models were developed using commercial software [10] to represent the behavior in a typical floor plate. Two areas of the building were chosen to be modeled as shown in Figure 2. Model 1 (Figures 3 and 4) represented the structure spanning onto the masonry façade and the transition zone where the direction of the slab span changes at the corner of the building. The beams are 9 m long on this side of the building. Model 2 (Figures 5 and 6) represented the slightly longer 10 m span beams at the rear of the building.

The larger, 9 m span model 1, is shown in Figures 3 and 4. The proposed protection arrangement is shown in Figure 3 in the region modeled.

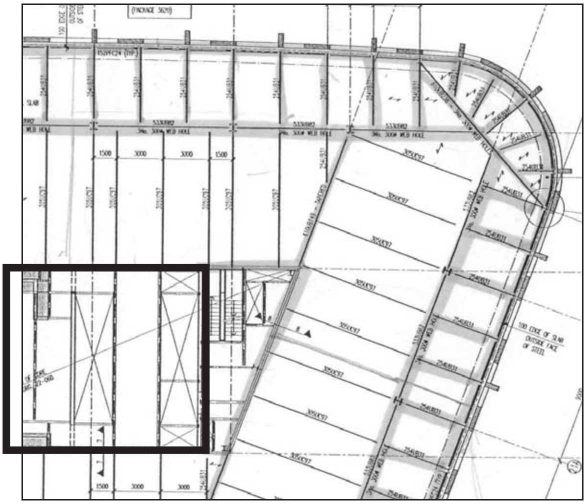


Figure 3. *Proposed protection arrangement.*

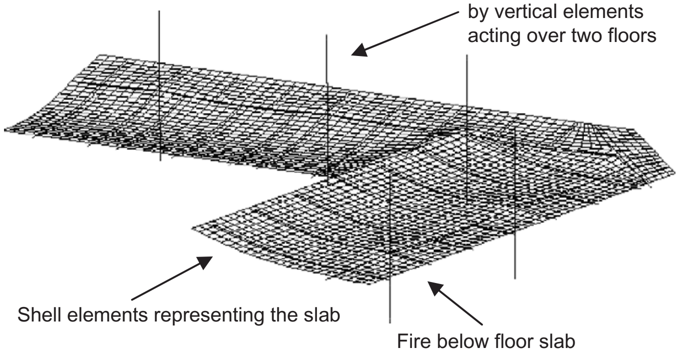


Figure 4. *The 9 m span global model 1.*

The primary, edge, and short secondary beams are protected leaving the main secondary beams bare. The columns and the steelwork in the fire fighting shaft and the core are fully protected.

The 10 m span model 2 represented a structural bay 9 m × 10 m and is shown in Figures 5 and 6.

The material properties assumed in both FE models are given in Table 1; full degradation of the stress–strain curves with temperature was allowed. Values of thermal expansion for steel and concrete were also taken from the

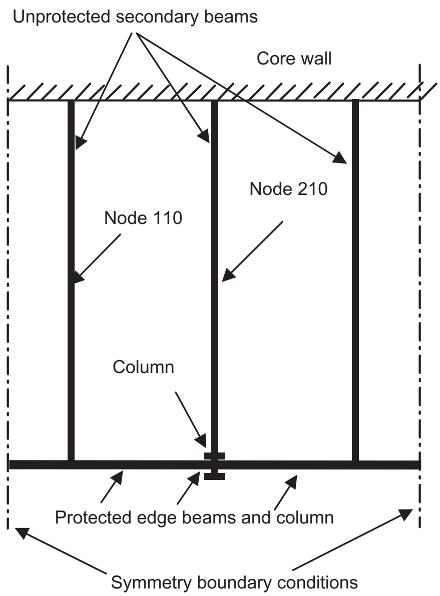


Figure 5. *Schematic plan view of the 10 m span global model.*

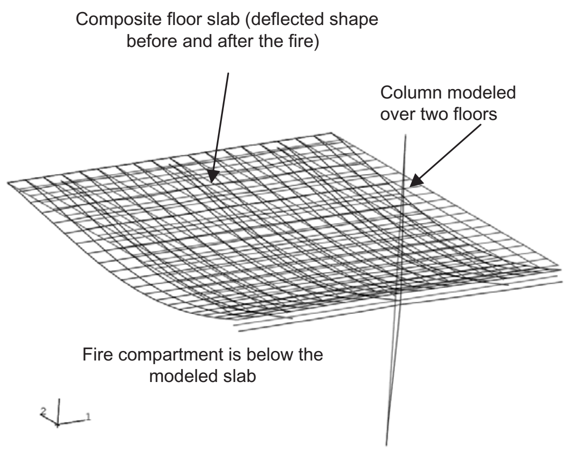


Figure 6. *The 10 m span global model 2.*

appropriate Eurocodes (EN 1993-1-2 for steel and EN 1991-1-2 for concrete).

In accordance with BS 5950 Part 8 [13], during a fire, the partial factors to be applied to live and dead loads are 0.8 and 1.0, respectively. These factors

Table 1. The material models.

Material	Grade	Model
Lightweight concrete (slab)	C30	Eurocode 2 [11]
Reinforcing mesh	S460	Eurocode 2 [11]
Steel (frame)	S275	Eurocode 3 [12]

were applied to the characteristic dead and live loads assumed by the structural engineer for the cold design. The load assumed to act over the floor slab of a typical office floor in the models was 7.85 kN/m^2.

The boundary conditions assumed in the FE models were as follows:

- Columns were fixed at their base and restrained in the horizontal directions but free to deflect vertically at the top. These boundary conditions simulated the continuity of the columns at the base of the structure and at the top of the columns.
- Slab and beams were fully fixed at the core wall.
- Symmetry boundary conditions were applied along the sides of the model parallel to the secondary beams.
- In the 9 m span model (Model 1), the short secondary beams were assumed to be axially restrained by the masonry façade, but rotationally free.
- The 10 m span model assumed symmetry boundary conditions on both sides perpendicular to the core (see Figure 5). These are very conservative, as it assumes that the floor plate is an infinitely long rectangle, which is significant because research has shown that square panels, supported on all four sides by protected composite beams, are much stronger than rectangular panels that are effectively supported on two sides only and span in one direction. In other words, the square arrangement allows 2D membrane action whereas the rectangle relies on 1D catenary action similar to the load carrying mechanism in beams at large deflections.

In both models, four-node shell elements were used to represent the slab. Two-node beam elements were used to represent the beams, columns, and slab ribs. Each element was associated with its appropriate section properties and material characteristics.

The columns were modeled on the fire floor and the floor above (see Figures 4 and 6). Slab shell elements were not connected to columns because stress can 'flow' around the column as a result of slab continuity and the models represented this. Slab elements were connected to beam elements using constraint equations between the beam and slab representing full composite action.

In all cases, the model elements were fully geometrically nonlinear and were also associated with nonlinear material properties.

Structure Temperatures

The design fires were selected as full flashover conditions on one floor only assuming the sprinkler system had failed.

The amount of ventilation available to an office fire can vary depending on the amount of glazing that breaks during the fire. Modern toughened double glazing systems may not break as readily as single panes of ordinary glass. It was therefore proposed that two levels of ventilation would be modeled, one with a high opening factor resulting in a relatively short duration fire with high maximum temperatures and one with a lower opening factor resulting in a greater duration but lower maximum temperatures.

It was agreed with the approving authorities that the following fires would be used for modeling purposes:

- 'Short hot' fully flashed over whole floor fire with a peak temperature of 1200°C and a duration of 30 min (assuming 100% of the available glazing breaks).
- 'Long cool' fully flashed over whole floor fire with a peak temperature of 950°C and a duration of 145 min (assuming 25% of the available glazing breaks).
- Standard fire of 90 min.

The fires are illustrated in Figure 7. It can be seen that the two levels of ventilation gave two very different fires – one long relatively cool fire, one short relatively hot fire.

The steel temperatures as a result of the fire scenarios were calculated using the lumped mass heat transfer equations [12] in Eurocode 3, Part 1.2. These relatively simple equations can be solved using a spreadsheet and allow average temperatures of the steel section to be calculated. The steel temperatures are illustrated in Figures 8–10 for each design fire. A 1D FE heat transfer model was used to establish the gradient through the depth of the slab in response to each design fire.

The slab temperatures in response to the standard fire exposure were represented in the structural model by an equivalent mean temperature and associated linear gradient acting at the centroid of the slab. The equivalent heating regime was calculated by establishing the stress state of the slab in response to the actual heating regime and then applying an equivalent stress state in the form of a mean temperature and linear gradient. This concept is explained in more detail by Usmani [8] and hence not repeated here.

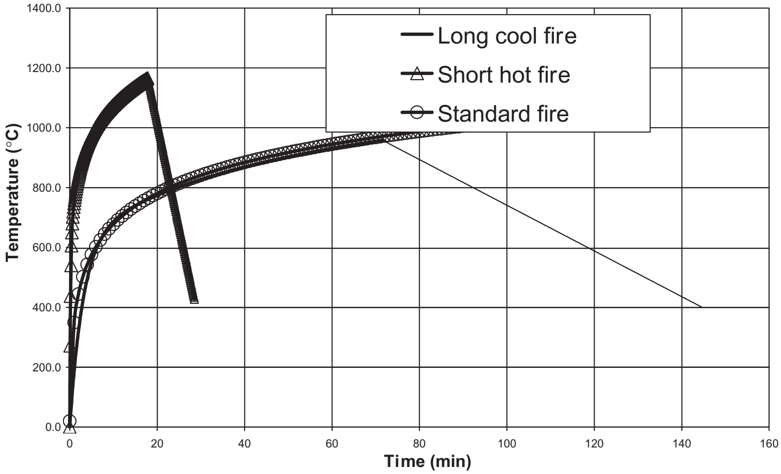


Figure 7. *Design fires for the structural analysis.*

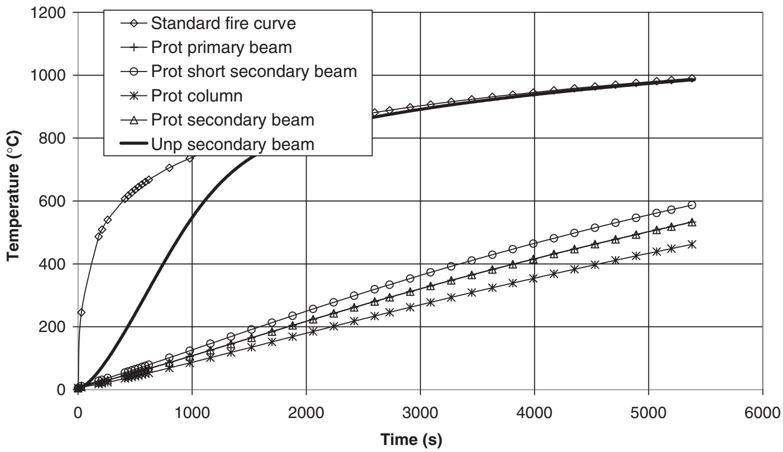


Figure 8. *Steel temperatures used in the FE model with standard fire exposure (prot = protected, unp = unprotected).*

The slab temperatures in structural models with the 'design' fire exposures including cooling were modeled explicitly. The actual temperature gradients through the depth of the slab were modeled and are illustrated in Figures 11 and 12. The temperature of the slab during the cooling phase of the 'design' fires cannot be represented by the mean temperature and linear gradient approach used for the standard fire case.

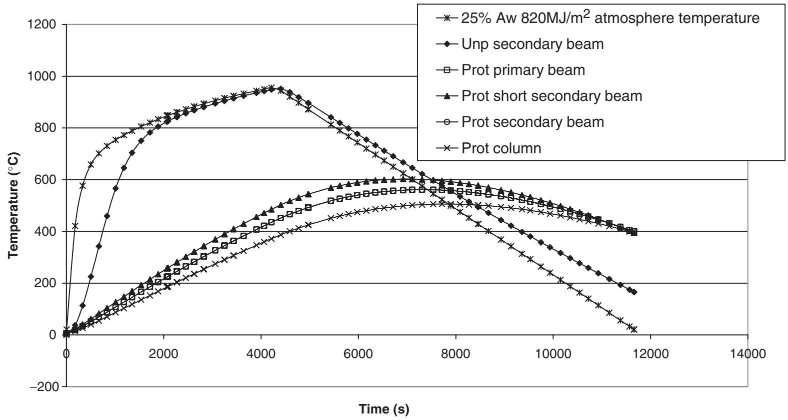


Figure 9. *Temperature histories in the steelwork for the design fire with 25% of the available glazing on one floor having failed providing ventilation (prot = protected, unp = unprotected).*

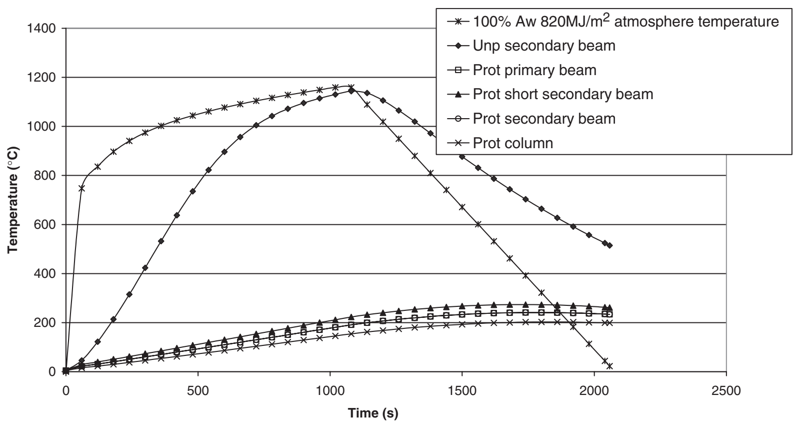


Figure 10. *Temperature histories in the steelwork for the design fire with 100% of the available glazing on one floor having failed providing ventilation (prot = protected, unp = unprotected).*

For each structural analysis, it was assumed that there was no gradient through the depth or along the length of the steel beams because in composite frames the most important gradient is that between the slab and the protected or unprotected steel beams. The gradient over the depth of the beam is much less important because it is very small in comparison. The columns on the fire floor were also uniformly heated in the models because

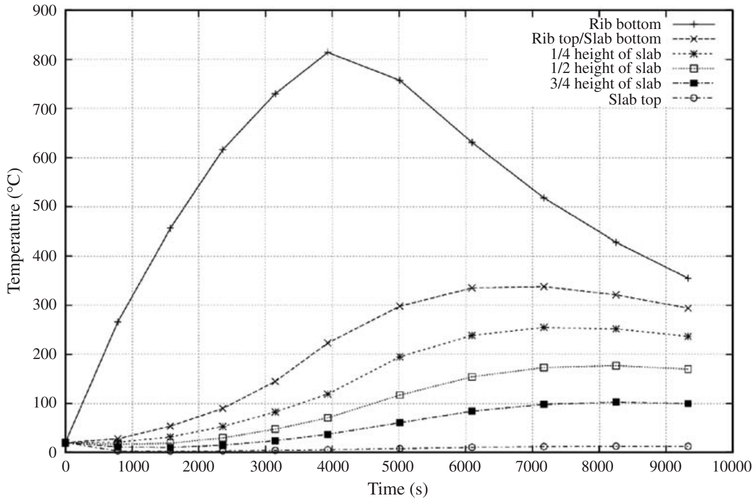


Figure 11. *Temperature histories in the concrete slab for the design fire with 25% of the available glazing on one floor having failed providing ventilation.*

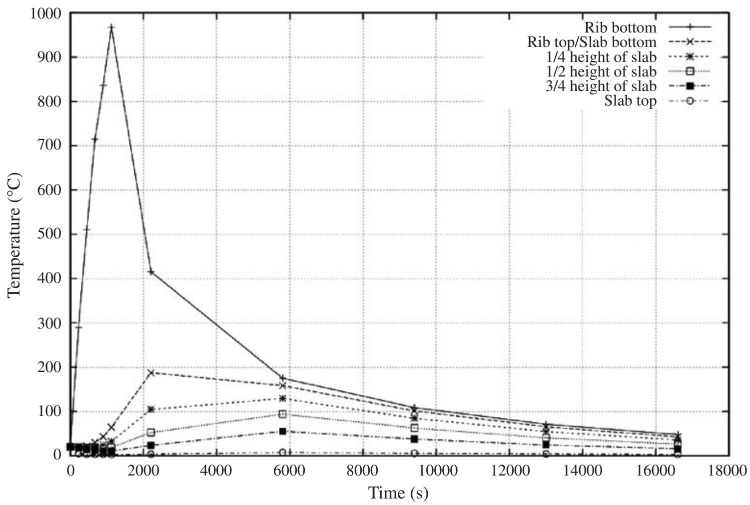


Figure 12. *Temperature histories in the concrete slab for the design fire with 100% of the available glazing on one floor having failed providing ventilation.*

they would be exposed to heating on all four sides during a flashover fire. The slab was assumed to be at a uniform, through depth gradient over the whole compartment as a result of the whole compartment having flashed over.

RESULTS WITH GLOBAL MODEL 1 AND 9 m SPANS

Proposed Structure with Unprotected Secondary Steel Beams in Response to the Standard Fire

A contour plot of the deflection at the end of heating is shown in Figure 13 for the case where the slab and beams were axially restrained by the masonry wall. The greatest downward displacement is near the midspan of the unprotected secondary beams as expected. The position of the columns is clearly visible. The structure is very stiff at the corner of the building where the short, protected secondary beams make a stiff, closely spaced grid. There is very little displacement in this region.

The midspan displacement of a typical unprotected secondary beam is shown in Figure 14. It is plotted against unprotected secondary beam temperature. The rate of deflection is very linear, similar to deflection plots from the Cardington tests. Runaway failure (a rapid increase in the rate of deflection) is not observed.

The axial force at midspan of a typical secondary beam is shown in Figure 15. As a result of the live load, the beam is in tension initially. Then the unprotected steel expands against the surrounding structure producing compressive forces very rapidly until 140°C when the steel reaches its first yield. Beyond this temperature, the axial force declines in compression with an increasing loss in material strength and stiffness until at the end of heating, the axial force is effectively zero. At this stage, the slab is therefore carrying load in membrane action.

The total strains (thermal + mechanical) in the slab at reinforcement level are plotted in Figure 16 for the Y- (=2) direction. Compressions have

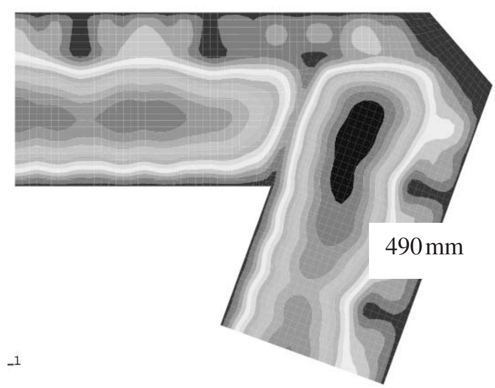


Figure 13. *Contour plot of deflection at the end of heating.*

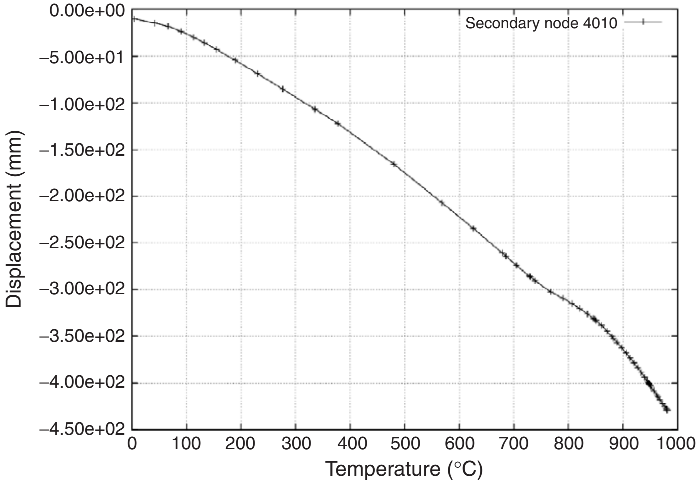


Figure 14. *Midspan deflection of a typical unprotected secondary beam (secondary node 4010 = node number at midspan of the secondary beam in the model).*

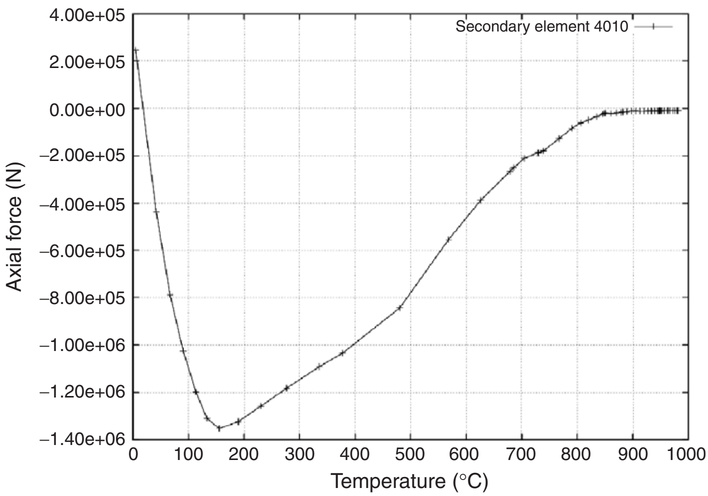


Figure 15. *Axial force at midspan of the unprotected secondary beam.*

negative values and tensions positive values. In general, the slab is in compression or low tension.

Thermal strains will account for about 0.1–0.3% of the total strain values. There are regions of relatively high tension (2–3%) near the core as expected. These are mainly as a result of the hogging moment at this

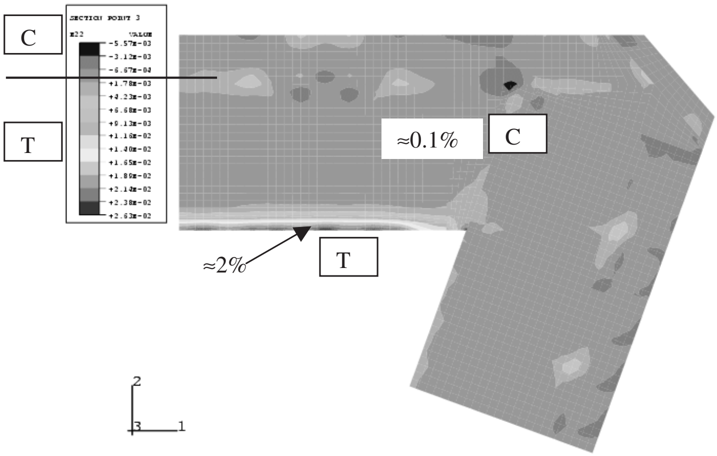


Figure 16. Strain in the Y- (=2) direction at the level of the reinforcement in the slab after 90 min of the standard fire exposure. C = compression, T = tension.

boundary. Any localized concrete cracking in this region would relieve hogging moments although strains will still be present after cracking as the deflecting slab pulls on the supports. The ability of the core connection to cope with the conditions during a fire was tested by a detailed connection model briefly described later in this article.

Fully Protected Structure in Response to the Standard Fire

During the design process, a direct comparison was made between the structural behavior observed in the model of the proposed design (reported in the previous section) with that which would normally be designed as a result of the recommendations in the Building Regulations i.e., all structural steel protected.

Figure 17 is a contour plot of deflections at the end of heating when all structural steel is protected. The maximum deflection experienced is 390 mm. Most secondary steel beams deflect up to 200 mm. This is contrary to the common belief that a protected structure does not deflect. Note also that this deflection is in excess of the BS476 requirement for $L/30$ deflection limits for beams/floors. This can be compared to the design case with unprotected secondary beams where the maximum deflection is 490 mm and the midspan deflection of the unprotected secondary beams is about 450 mm (see Figures 13 and 14).

Therefore in terms of damage to the structure in the context of insurance, the traditional design approach and the proposed design result in identical

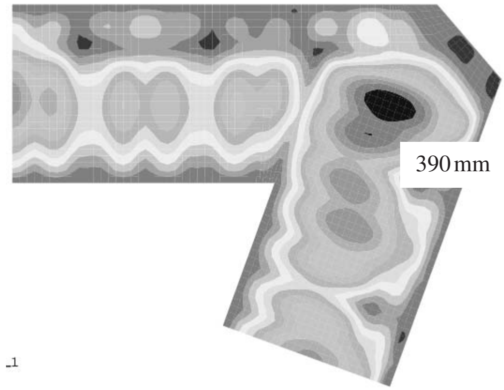


Figure 17. *Contour plot of deflection at the end of heating.*

structural member replacement measures after a fire of severity assumed in this model.

The heating regime in this standard fire analysis is based on the assumption that the protected steel will reach a maximum temperature of about 550°C at the end of 90 min. This is based on the UK requirements for fire proofing.

The deflection at midspan of a typical fully protected secondary beam is very linear i.e., the rate of deflection is not changing (see Figure 18). The same behavior was shown in Figure 14 when the beams were unprotected although the deflections were much greater. This suggests the structure is very stable. The uniform rate of deflection was also observed in the measurements made at Cardington during the fire tests.

The strains in the $Y(=2)$-direction are shown in Figure 19. Tensile strains along the core edge are in the region of 2%. The greatest tensile strains are around the column locations and at the core wall. The strains experienced in the slab when all beams are protected are very similar to the design case with secondary beams unprotected. It could be expected therefore that the slab would also experience local cracking in the fully protected case.

Significance of Results from Global Model 1 in Response to the Standard Fire with 9 m Spans

The comparative analyses have shown that the deflection and strain patterns in the composite slab are very similar for both protection arrangements. Therefore, it could be assumed that the damage to the structure would be similar in both cases. The midspan deflection of the floor

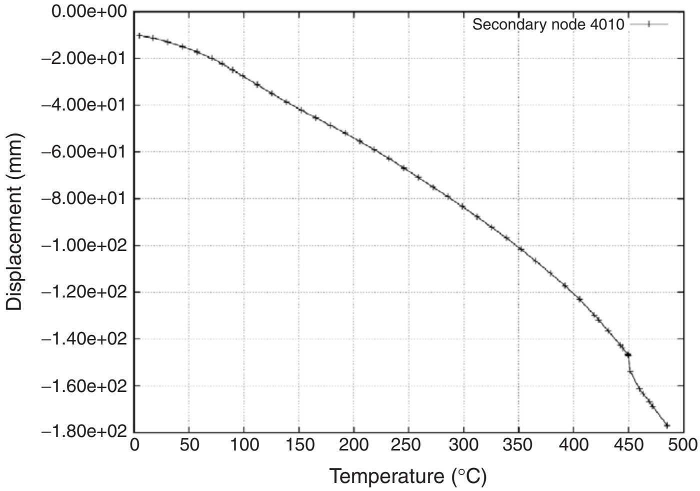


Figure 18. *Deflection at midspan of the secondary beam.*

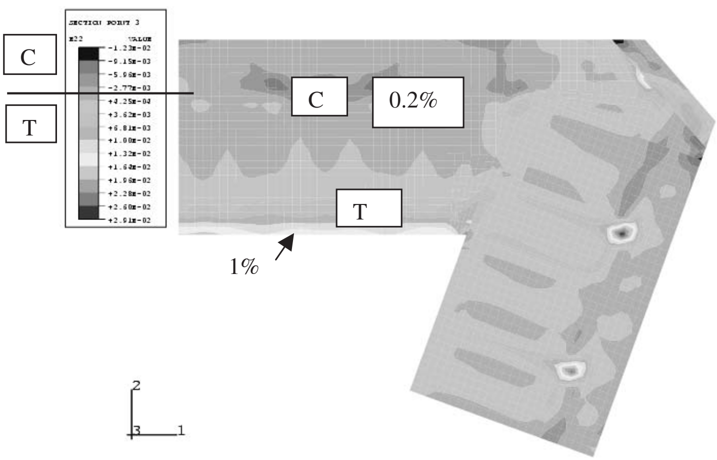


Figure 19. *Strain in the Y(=2)-direction at the level of the reinforcement in the slab. C = compression, T = tension.*

slab was 390 mm when all beams were protected and only ≈100 mm greater when the secondary steel beams were unprotected. Similarly, the strains predicted in the concrete slab near the core are ≈2% in both protection cases.

The FE models allowed approving authorities, insurers, and clients to see the likely damage, rather than relying on prescriptive guidance.

When quoting insurance premiums, insurers have traditionally had to guess the likely damage to structures in fully flashed over compartment fires because real structural behavior is vastly different from the standard furnace test. The modeling methodology provides invaluable information for all concerned.

It should be noted that the results of these models is for this particular building and in another structure with different spans and layout, the results of a similar comparative study may not be so similar. This type of design process must be carried out on a case-by-case basis. Generalized conclusions cannot be made.

RESULTS WITH MODEL 2 AND 10 m SPANS

Model 1 captured the most novel part of the building structure, including the masonry façade and the slab spanning in two different directions. However, the spans in this region are 9 m, whereas on the opposite side of the floor plate at the rear of the building, beams span 10 m. These larger spans generated greater thermal expansion and therefore greater midspan deflections. Larger tensions were observed at the slab to core interface and runaway failure was observed. This section therefore discusses the differing response of the frame at the rear of the building to the two parametric design fires and the design changes made to mitigate failure as a result of the observed results.

'Short Hot' Fire

Figure 20 shows the deflection at midspan (Node 210, see Figure 5) of the central secondary beam against unprotected secondary beam temperature during the 'short hot' fire. The deflection history is linear in the early stages of the fire but the rate increases as the temperature increases. At the end of heating (secondary beam temperature of 1150°C) and into the cooling phase of the fire, the deflection continues to increase because the slab is still undergoing heating due to the thermal lag concrete experiences when exposed to high temperatures. After 800°C, the deflection rate is constant and, if the analysis had been allowed to continue, the deflection would partially recover. As the steel beam experienced considerable yielding in heating, it cannot return fully to its original position.

Figure 21 shows the axial force at midspan of the central secondary beam (Node 210). As expected, the beam is in significant compression as it expands against its supports during heating. The steel beam yields at a temperature of about 150°C. Beyond this, the beam attracts no more compression and begins to reduce in compression as the material strength

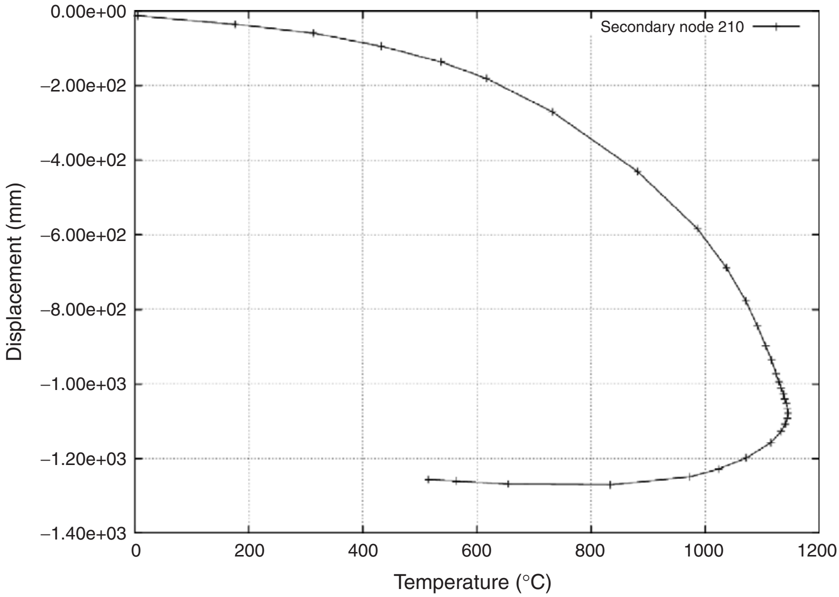


Figure 20. *Deflection at midspan of the secondary beam at the center of the model.*

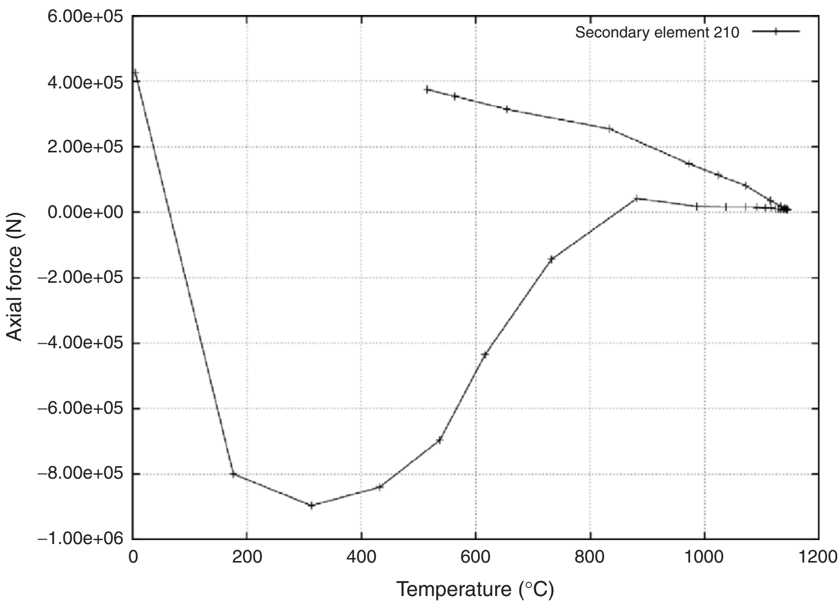


Figure 21. *Axial force at midspan of the secondary beam.*

decreases with increasing temperature. The ultimate yield of the steel beam takes place at about 500°C.

Between 800 and 1100°C, the beam is in tension, showing that the beam is carrying load in catenary action. This is a sign of impending runaway failure because the structure has to utilize almost all of its strength. Therefore, failure is assumed to occur for this design case.

During the cooling phase of the fire, the steel begins to recover some of its strength and moves into tension as the slab (which is still undergoing heating as a result of thermal lag) pushes the steel beam downward and the steel tries to contract into its original position.

A plot of strain at the level of the reinforcement in the slab in the *Y*-direction is shown in Figure 22 at the end of heating. The greatest total strains are at the core edge and around the column (the regions of greatest hogging). Strains at the core were predicted to be about 2% while strains at the column were about 4%. This is high and, although the concrete will crack and the mesh is quite ductile, this was not deemed acceptable for design. The unacceptable result was addressed by protecting the secondary beams between the columns to reduce deflections and the analysis repeated. Note that the structural model was conservative because the slab was assumed to be infinitely long; in reality it is connected to the rest of the floor plate.

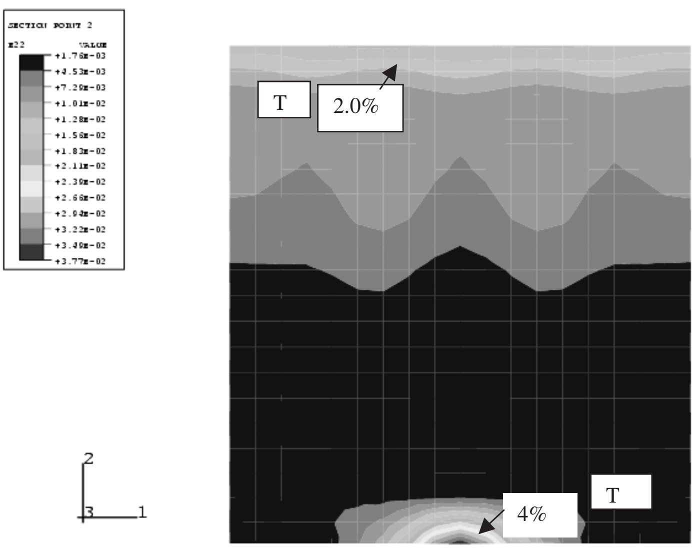


Figure 22. *Strain in the Y-direction at the level of the reinforcement in the slab at the end of heating.*

'Short Hot' Fire with Secondary Beams Protected between Columns

The deflection at midspan of an unprotected secondary beam is shown in Figure 23 for the 'short hot' fire when secondary beams between columns were protected. The deflection rate is very uniform and the structure behaves well. Deflections are reduced by about 50% compared to the case with all secondary beams unprotected (see Figure 20). The total strains in the $Y(=2)$-direction are plotted in Figure 24. The tensile strain at the core wall is now $<1\%$, compared to 4% when the secondary beams between columns were unprotected. The strains are also lower close to the core wall.

Therefore, by protecting steel beams connected directly to columns, failure is no longer observed to occur, even with all other secondary steel beams unprotected.

'Long Cool' Fire

Midspan deflection of the unprotected secondary beam is shown in Figure 25 with all secondary beams unprotected. The deflection rate increases rapidly after temperatures of 600°C. Runaway failure was observed and the analysis failed to converge as a result of this at 780°C.

This result is not totally realistic because the symmetry boundary conditions in the model assume an infinitely long slab. If the whole floor

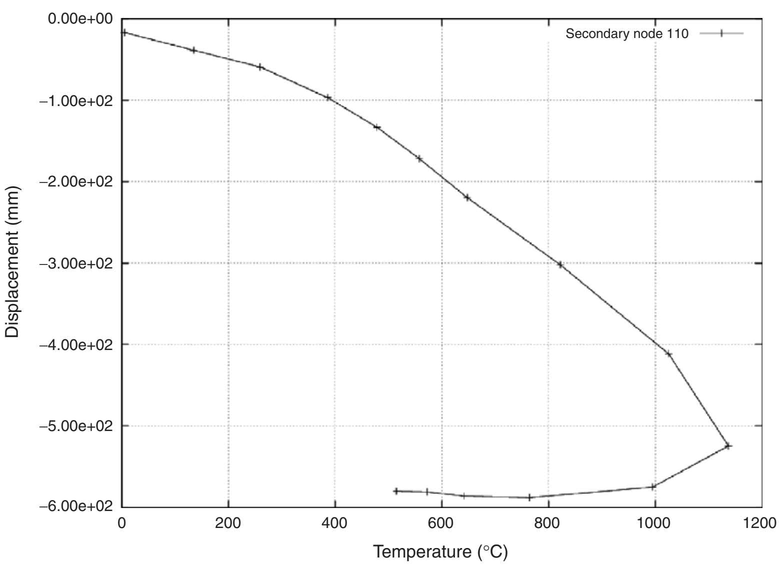


***Figure 23.** Deflection at the midspan of the secondary beam.*

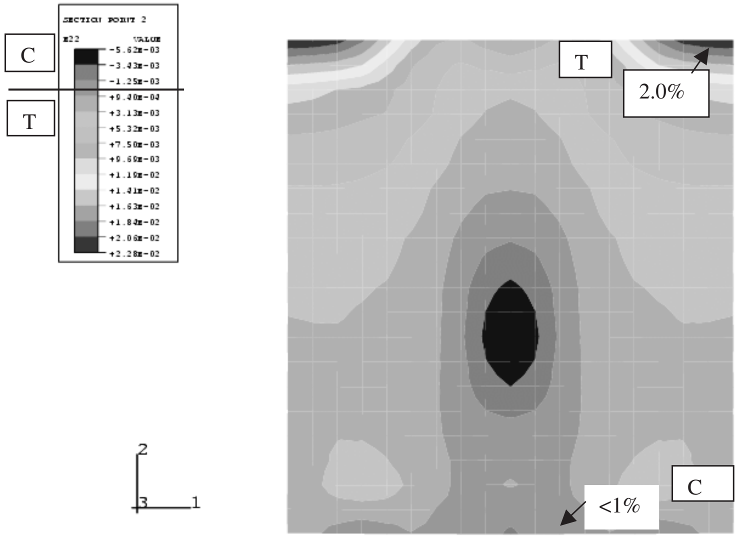


Figure 24. *Strain in the Y-direction at the level of the reinforcement in the slab at the end of heating.*

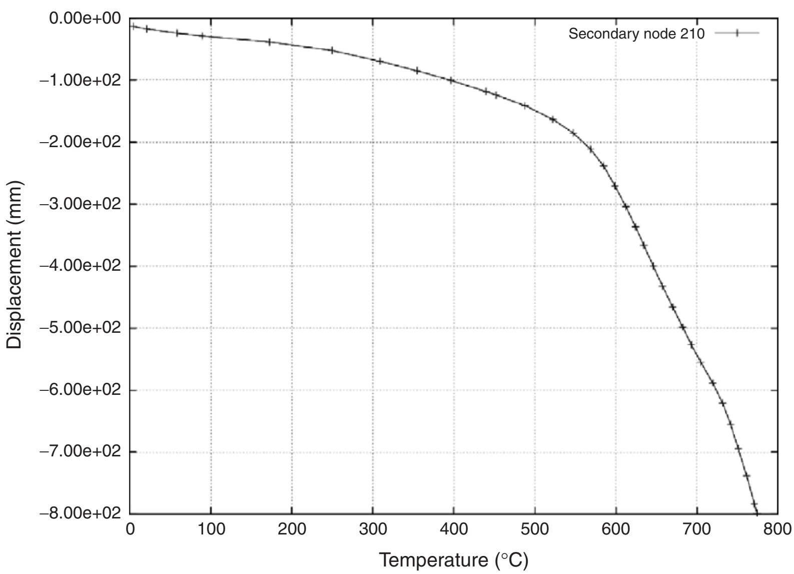


Figure 25. *Runaway deflections of the secondary beam midway through the heating phase of the fire.*

slab had been modeled, the failure is unlikely to have been observed. However, as the conservative FE model did show failure for the 'long cool' design fire, the secondary beams between columns were protected and the analysis was repeated.

'Long Cool' Fire with Secondary Beams Protected between Columns

The midspan deflections of the protected and unprotected secondary beams in the revised model are shown in Figure 26. When the secondary beams between columns are protected, there is no evidence of runaway failure. Almost all deflections are associated with thermal effects. At an unprotected steel temperature of 800°C, the protected steel beam would have reached about 150°C and buckled near the support, allowing thermal expansions to be absorbed in downward deflections. This also increases the rate of deflection in the unprotected secondary beams.

The strains at the level of the reinforcement are shown in Figure 27 at the end of heating in the $Y(=2)$-direction. Maximum total tensile strains of about 4% occur in regions along the core edge. Again this is high and is partly due to the fully fixed boundary condition assumed at the boundary of the global model. A local model of the core connection was built to check the forces induced in the rebar and to ensure that failure did not occur.

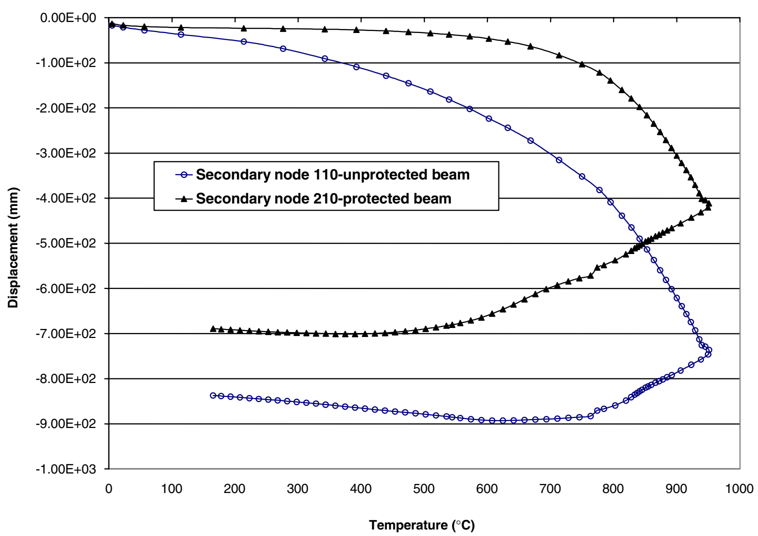


Figure 26. *Midspan deflection of a protected and unprotected beam in the 'long cool' fire (see Figure 5 for the location of nodes 110 and 210). (The color version of this figure is available on-line.)*

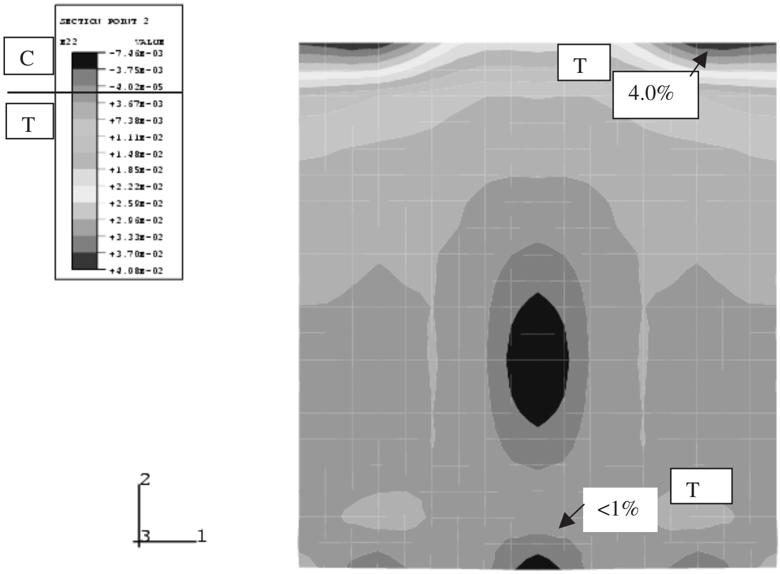


Figure 27. *Strain in the Y-direction at the level of the reinforcement in the slab at the end of heating.*

Significance of Results from Global Model 2 with 10 m Span

The 'long cool' fire is worse for this particular structure than the 'short hot' fire. When all the secondary steel beams were left unprotected, runaway failure was observed for the 'long cool' fire only.

Model 2 was conservative as a result of the symmetry boundary conditions assumed because the real structure is not infinitely long perpendicular to the secondary beams. The aspect ratio of the floor plate is much smaller in reality.

As a result of the structural behavior seen in the 10 m span model (Model 2), secondary steel beams between columns were protected to allow tensile membrane action at high temperatures and large deflections to develop in a 2D manner rather than in a 1D catenary action.

Strains in the rebar at the core to slab interface were found to be high as a result of the boundary conditions assumed in the global model. In order to ensure that the tensile membrane action can be supported without pulling out the connections to the core or rupturing the rebar, an explicit connection model was developed.

CONNECTION MODELING AT THE FLOOR TO THE CORE BOUNDARY

The global FE modeling of the composite floor slab showed that a number of secondary steel beams could be left unprotected as they are not essential to the stability of the floor slab during a fire. However, the global FE model used had a number of simplifications:

- It did not consider concrete cracking explicitly.
- It treated the reinforcement mesh as a smeared layer in the thickness of the shell elements representing the slab. This smeared mesh did not model localized fracture of the reinforcement.
- It assumed connections are perfect pins or fixed supports with no account of material degradation.

The global FE analysis appeared to indicate that high loads/deformations were generated in the beam connections and reinforcement around the core. As the global analysis did not consider the connection details into the core accurately, a local explicit model was developed to investigate whether the reinforcement between the slab and the core and the steel beam to core connection would be able to remain intact (i.e., not rupture) if some of the secondary beams were unprotected. The local model included the worst case of credible design fire defined by the global FE analysis as the 'long cool' fire.

Connection Model Description

The local model represented a 10 m span section of an office floor between columns. The use of symmetry conditions in the model allowed the extent of the model to be limited to a $5\,m \times 4.5\,m$ area. The model is shown in Figures 28 and 29. This connection was chosen because it was the connection under greatest load from the results of the global models.

In the local model, the back of the core wall was modeled as rigid and fixed. The 10 m 305UC97 steel beams were modeled with nonlinear shell elements. The protected primary beam was given fixed ends, whereas the unprotected secondary beam had a bolted connection detail with the core wall.

The floor slab was modeled as 130 mm thick with a reinforcement grid of T6 bars at 200 mm spacing (0.22% reinforcement). T10 bend-out bars at 300 mm spacing connect the core wall to the slab. These were represented in the model by small beam elements, which allowed the connection to fail.

The connection detail for the 10 m unprotected beam consists of a plate cast into the core wall with a fin plate and 3 M20 Grade 8.8 bolts connecting

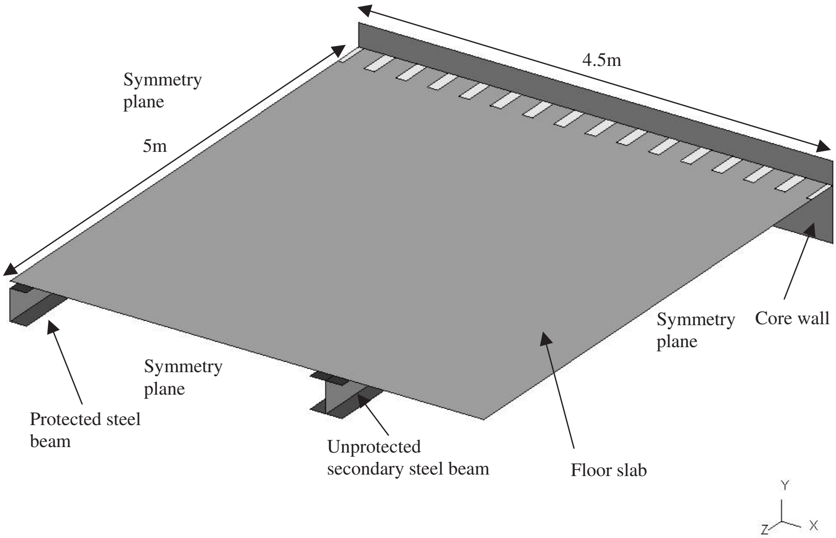


Figure 28. *The LS DYNA model.*

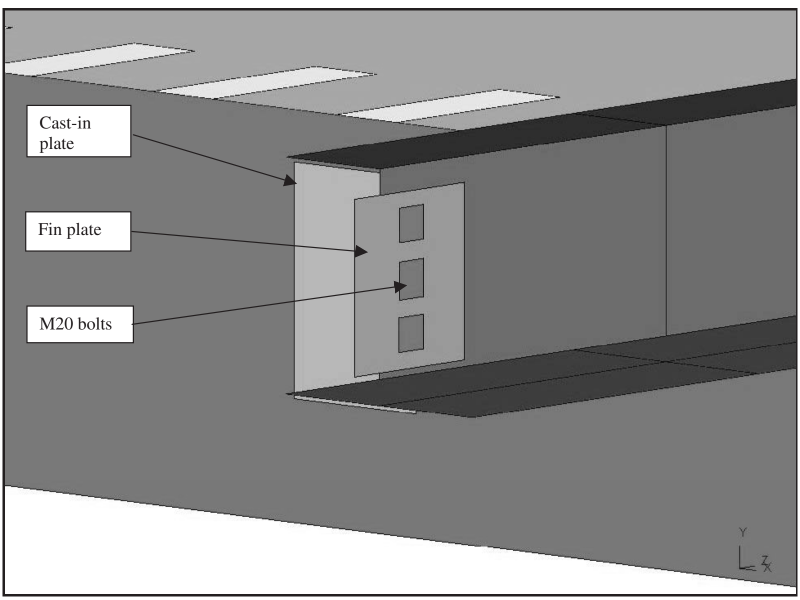


Figure 29. *The connection detail in the LS DYNA model.*

the fin plate to the web of the beam. The bolts are situated within 22×122 mm long slotted holes in the fin plate. In the model, the bolts were represented by small rigid patches on the fin plate and web, which were connected by small beam elements. All of the characteristics associated with bolt deformation and failure, movement in the slotted holes, and bearing failure were represented in the properties given to the small beam elements.

The analysis was carried out twice, once with bare steel connections and once with protected connections. The extent to which the fire protection was modeled included the cast-in plate, the fin plate, and the final 225 mm of the secondary beam flange and web. All of the material properties and temperature–time histories were the same as those in the global models for the 'long cool' fire. The connection temperature was assumed to equal the protected secondary beam temperature.

Connection Analysis Results – Unprotected Connections

The connection model showed that as the steel beams and concrete floor slab started to expand, the floor slab bowed downward and the unprotected steel beam rotated downward at the connection detail as the bolts slid within the slotted holes. At an unprotected steel temperature of about 370°C, the bottom flange of the beam made contact with the core wall.

The web around the bolts was shown to fail at an unprotected steel temperature of about 700°C, as shown in Figure 30. At about the same time, a buckle began to appear in the bottom flange of the protected primary steel beam near to the fixed end. This buckle continued to grow, reaching its peak at a protected steel temperature of about 550°C.

Over the duration of the event, the concrete floor slab developed some cracking, as shown in the plot of maximum principal strain (i.e., tensile strain) in Figure 31. This shows the regions of greatest tensile strains.

Between unprotected steel temperatures of 800 and 930°C, a couple of the T10 bend-out bars around the unprotected secondary steel beam were predicted to fracture (due to excessive tensile strain). In addition, over the duration of the fire event, many of the other T10 bend-out bars between the unprotected secondary steel beams came close to fracture.

Since the web of the unprotected secondary steel beam failed at the connection, this steel beam was unable to support the floor slab after the fire. Therefore, the floor slab was required to span between the two protected secondary beams and/or between the core wall and the opposite protected edge beam. This was considered to be an acceptable level of deformation in the cooling phase of the fire.

The cracking predicted in areas of the concrete floor slab was relatively minor and in these areas the reinforcement grid would be expected to

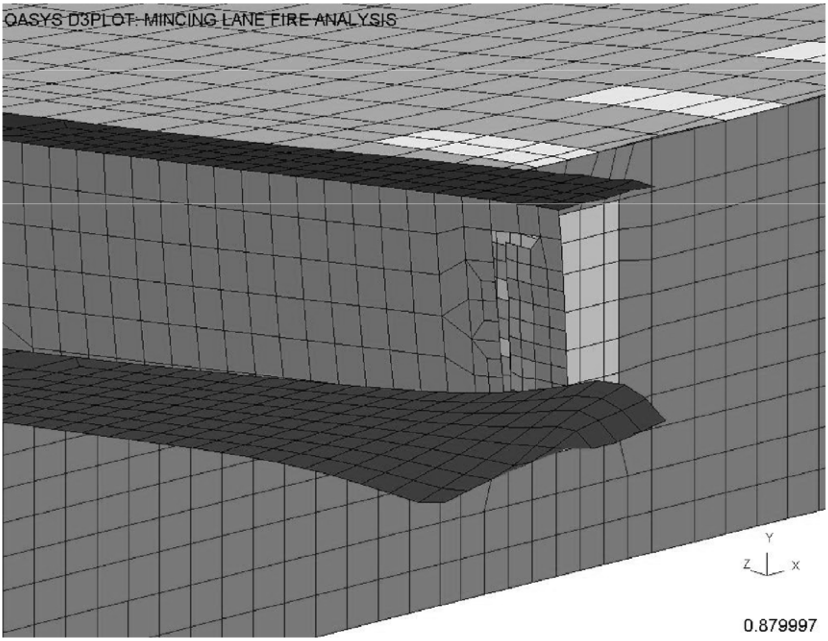


Figure 30. *Web fracture around the bolts.*

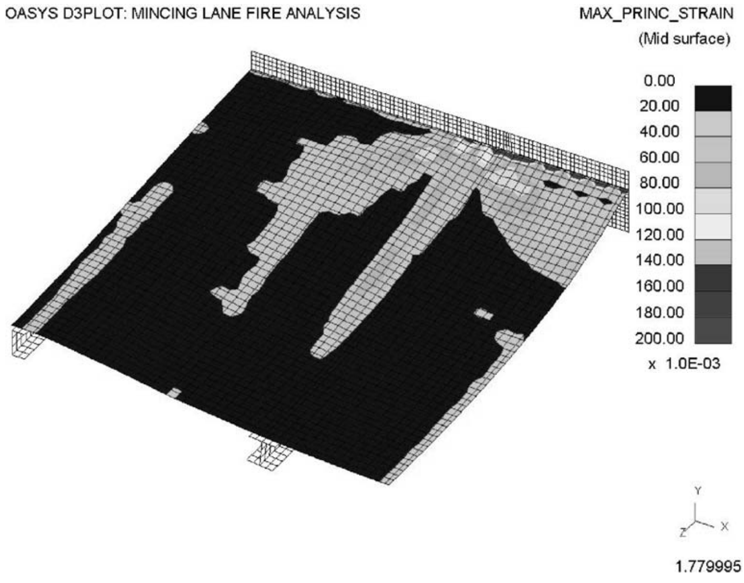


Figure 31. *Principal strains.*

maintain the structural integrity of the floor slab. In areas where the tensile strain was significant, the reinforcement grid could be expected to fail. The only areas with significant tensile strain in the floor slab were next to the core wall, between the two unprotected secondary beams. Therefore, it was assumed that the concrete floor slab would not retain a structural connection with the core wall in the region between the unprotected secondary beams.

However, the rest of the floor slab was predicted to maintain its structural integrity (with its reinforcement grid) and was able to span between the two primary beams and between the core wall and the opposite primary beam.

The analysis was repeated with protected connections. The fire protection significantly reduced the damage to the secondary beam and connection, such that they could be assumed to retain their structural integrity. There was also a corresponding reduction in the cracking of the concrete floor slab and the failure of the T10 reinforcement bars. The analysis indicated that only one T10 reinforcement bar (directly over the secondary beam) might fracture. Therefore, the connections to the core were protected on site.

CONCLUSIONS

This article provides a snapshot of information and analysis to demonstrate that the passive fire protection arrangement for an office building in London satisfies the appropriate functional requirements of the Approved Document B of the Building Regulations, UK. It compares the performance of a prescriptively designed structural frame with the performance of a structural frame having reduced passive fire protection in response to credible design fires, all as part of a research approach used to satisfy the stakeholders of a structural fire engineering solution.

A detailed FEA of the structure with a standard fire and credible design fires was carried out to determine the deflections and forces in the structural elements.

A direct comparison was made between the structural behavior for a proposed design with secondary steel left unprotected and structural behavior with all steel fire protected, as would be the case in a traditional prescriptive design. The results for the fully protected design were similar to those for the proposed design and clearly showed that any fire protection on the secondary beams was redundant.

The comparative analyses have shown that the deflection and strain patterns in the composite slab are very similar for both protection arrangements. Therefore, it could be assumed that the damage to the structure would be similar in both cases.

The comparative study was invaluable in the approvals process because the fire brigade, insurers, and approving authorities could quantify the differences in response between the prescriptive design they would normally approve and the performance of the proposed design with some bare steel.

A conservative model of the 10 m span bays in the structure showed that failure could occur in this case if all secondary steel beams were unprotected. This is due to the span length, reliance on 1-way catenary action instead of 2-way membrane action and the conservative model used. Detailed analysis of both global and local models resulted in a design solution that relied on the development of tensile membrane action in square panels between protected beams. Therefore, it was proposed to protect specific beams – those spanning directly between columns and connections into the concrete core.

This type of design should be carried out on a case-by-case basis and results in this article are applicable to this building design only.

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