Structural Behaviour in Fire and Real Design

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Structural Behaviour in Fire and Real Design

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ABSTRACT

A great deal of understanding into the behaviour of composite steel-concrete structures in fire has been developed since the Cardington frame fire tests (UK) 1990s. This has now been broadened so that structures in fire design 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 behaviour in fire. Since 9-11 robust engineering solutions where the response of the building to an event like fire is known are in great demand. The basics of structural mechanics at high temperatures can be used in design to understand many structures with the aid of computer modelling.

This paper provides a direct comparison between the structural response of an 11-storey office building now constructed in the city of London, when designed in a prescriptive manner, with applied fire protection on all the 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 the fire limit state can be maintained in specific cases without relying on passive fire protection.

This paper contributes to the field of structural fire engineering by extending the research work previously conducted by the authors[1] 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 response in 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 with the necessary stakeholders in this project.

Keywords: Fire engineering, design fires, structural response, thermal expansion, performance based design, prescriptive design, approvals process.

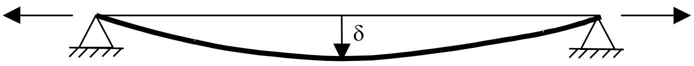
INTRODUCTION

Recent research in the field of structures in fire has been used to provide a robust design solution to the passive fire protection arrangement at an 11-storey office building in London, UK. Detailed Finite Element Analysis (FEA) allows engineers to examine the structural behaviour of a composite steel frame as it continues to support loading at the fire limit state. In some cases this type of analysis permits a reduction in the number of steel beams that require passive fire protection, whilst maintaining structural stability and compartmentation. This form of analysis also highlights areas where the structure is less robust during a fire and additional fire protection or structural measures that may therefore need to be introduced. This information is particularly relevant for tall building design since 9-11. Robust structures at ambient temperatures may not necessarily be robust when exposed to fire. And building owner and occupier demands post 9-11 means the ability to quantify real response has become an essential part of the design and approvals process.

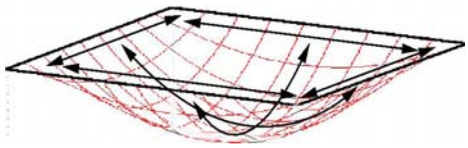
The Cardington Frame fire tests[2] in the UK in the 1990s provided a wealth of experimental evidence about how whole frame composite steel-concrete structures behave in fire. The Cardington Frame survived a number of full scale fire tests despite, in most cases, having no fire protection on any of the 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 behaviour in the standard fire resistance test. Engineers have always recognised that whole frame structural behaviour in fire cannot be described by a test on a single element. However, it is only in relatively recent years since the Broadgate Phase 8 fire in London, UK and the subsequent Cardington frame fire tests[2] that researchers have fully investigated and understood the behaviour of whole frame composite steel-concrete structures in response to fire.

The main conclusions of the tests and the subsequent research projects[2-5] were that composite framed structures possess reserves of strength by adopting large displacement configurations with catenary action in beams and tensile membrane behaviour in the slab[3-5] (see schematic representation in Figure 1). 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[3]. Large deflections were observed as not being a sign of instability and local buckling of beams in fact 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[2] believe that gravity loads and strength would have been the critical factors near impending failure.



(a) 1D Catenary action in beams



(b) 2D Tensile membrane action in slabs

Figure 1 Catenary action in beams and tensile membrane action in slabs

An indeterminate structure such as a multi-storey frame is capable of transferring load through many alternate load paths. This is true at ambient and at the 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 different 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)[6] of the Building Regulations in the UK. In this context this means the compartmentation arrangements and passive fire protection to the structure are designed to maintain the stability of the structure for a reasonable period and limit fire and smoke spread to the floors above the fire floor.

FIRE RESISTANCE TESTING VERSUS WHOLE FRAME BEHAVIOUR

The fire resistance levels recommended in regulatory documents are based on the behaviour of single structural elements heated in a furnace. The fire resistance of the structural element is taken as the time to the nearest minute, between commencement of heating and when failure occurs. Periods of fire resistance are normally specified as ½ hour, 1 hour, 1.5 hours, 2 hours, 3 hours and 4 hours. This is known as the standard furnace test or the fire resistance test. 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[7] defines three criteria for insulation, integrity and stability that must be passed in order to achieve a fire resistance rating. For stability of load bearing horizontal elements of structure e.g. beams and floor slabs 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. L=clear span of the specimen under test and d = the distance from the top of the structural section to the bottom of the design tension zone.

These limits are known to be based on the size of the furnace and tend to be the maximum deflection that can be recorded without causing damage to the furnace.

Therefore in a code compliant building in the UK with all structural elements protected the floor can deflect up to L/20. For a 7.5m beam this equates to 375mm and for an 18m beam this is 900mm.

Due to its simplicity the furnace test misses vital structural phenomena found in the 3D behaviour of real buildings including;

- Large deflections and nonlinear geometry.
- 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.

THE BUILDING

The case study, presented here to compare and contrast performance based design and a "code compliant" structural design for fire, is an office development in London consisting of an 11-storey office, eight storeys above ground and three below.

The floor plate measures 40m x 60m (see Figure 2). There is a concrete core at the centre of the building containing services and escape stairs, which are also designed for fire fighting (provided with fire fighter dedicated lifts, a lobby/vestibule separating the stair from the accommodation, and within a 2 hour enclosure). 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 9m between the core and the main column line and then shorter beams (~2.5m long) span between the main column line and the masonry façade. Model 1 illustrated in Figure 2 captures this portion of the floor plate. To the rear of the building, primary and secondary steel beams span 10m between the core and the column line on the façade (captured by Model 2).

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

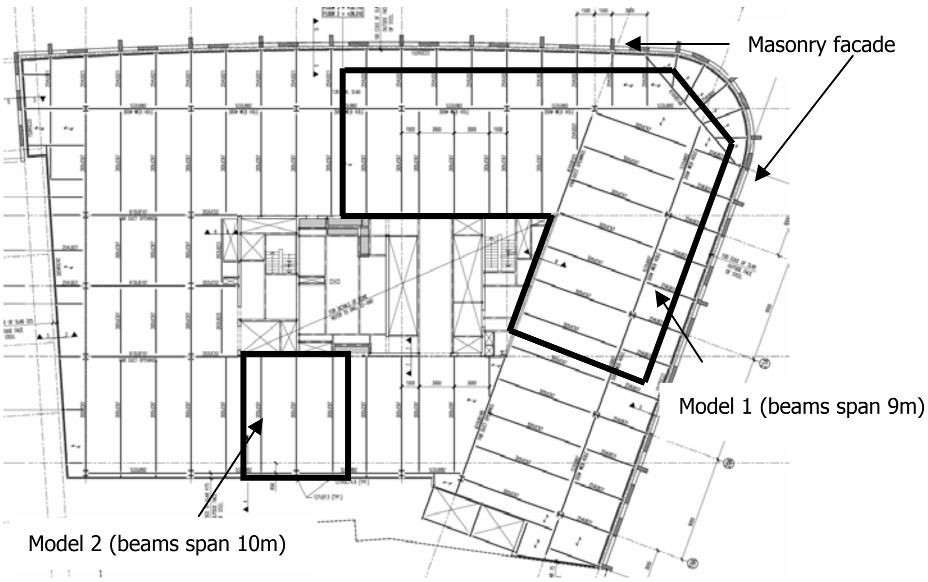


Figure 2 Plan of the office building showing the extent of the global FE models

FAILURE CRITERIA IN COMPOSITE FRAME STRUCTURES

In order to assess the data provided from a finite element analysis some means of defining failure criteria must be established. There is currently in the UK no Regulatory definition of failure.

The term 'failure' is not straightforward to define in the context of this type of analysis on the basis that, although a compartment fire may lead to large deflections of main and secondary beams, this is unlikely to cause structural collapse i.e. stability requirements can be met.

However for compartmentation large deflections could cause a breech of the separating function of the element such as the floor, or the escape cores.

On this basis the following were proposed to the Stakeholders (client, insurer, fire authority, building authority) as acceptance criteria:

- Stability of structure 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 failure of the floor system and pulling in of the columns.
- Horizontal compartmentation 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 (a fire rated shaft required in the UK in buildings with a floor 18m above fire service access level to provide a place of relative safety on each floor for fire fighters) 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 the FE code ABAQUS[8] (see Figure 2) to represent the behaviour in a typical floor plate. Model 1 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 9m long on this side of the building. Model 2 represented the slightly longer 10m span beams at the rear of the building. The larger, 9m span model, (model 1) is shown in Figures 3 and 4.

The proposed protection arrangement is shown in Figure 3 in the region modelled. 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.

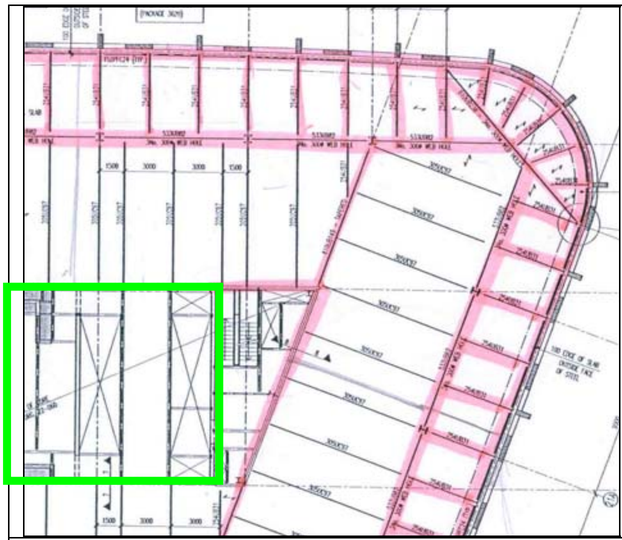


Figure 3 Proposed protection arrangement

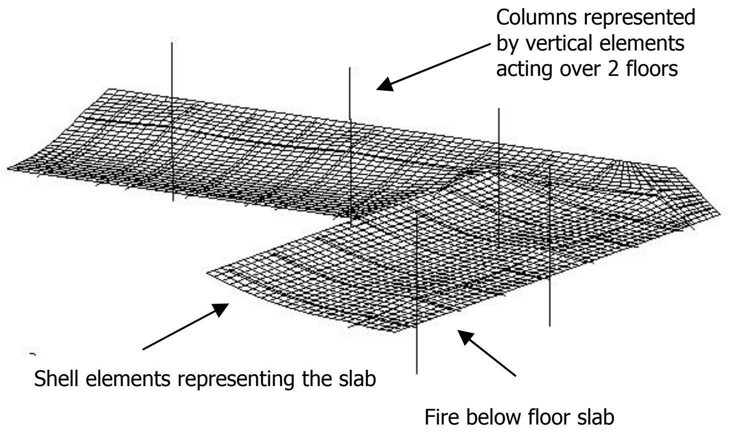


Figure 4 The ABAQUS 9m span global model

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

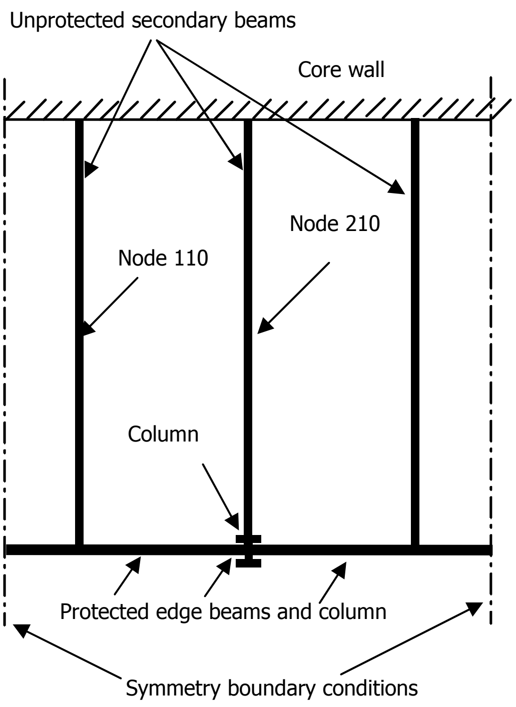


Figure 5 Schematic plan view of the 10m span global model

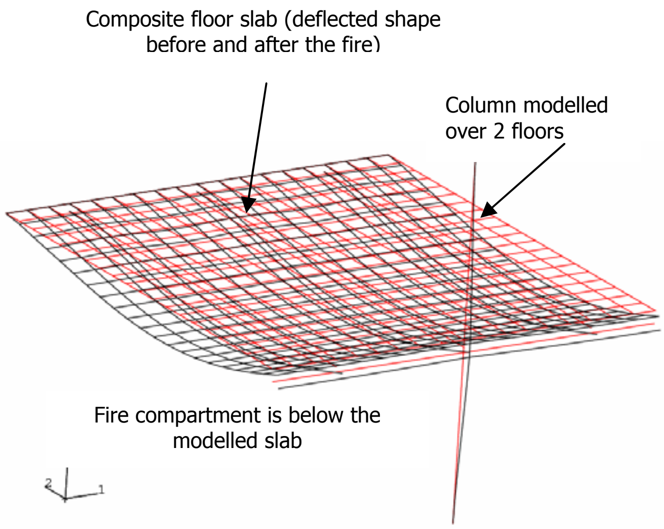


Figure 6 The ABAQUS 10m span global model

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

Table 1 The material models

Material	Grade	Model
Light weight Concrete (slab)	C30	Eurocode 2[8]
Reinforcing mesh	S460	Eurocode 2[8]
Steel (frame)	S275	Eurocode 3[9]

In accordance with BS 5950 Part 8[10], at the fire limit state, the partial factors to be applied to live and dead load are 0.8 and 1.0 respectively. These factors 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 7.85kN/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 downwards 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 9m span model (Model 1) the short secondary beams were assumed to be axially restrained by the masonry façade but rotationally free. In other models not described here, it was conservatively assumed that the masonry wall provided no restraint because the restraint stiffness and reliability of the connections to the masonry façade was unknown.
- The 10m span model assumed symmetry boundary conditions on both sides perpendicular to the core (see Figure 5). This was a very conservative assumption as it effectively assumes that the floor plate is an infinitely long rectangle and is significant because research has shown that square panels, supported on all four sides by protected composite beams are much stronger than rectangular panels which are effectively supported on two sides only and span in one direction. In other words the square arrangement allows 2D membrane action where as the rectangle relies on 1D catenary action similar to the load carrying mechanism in beams at large deflections.

In both models four noded shell elements were used to represent the slab. Two-noded 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 modelled 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 non-linear and were also associated with non-linear material properties.

Structure temperatures

It was assumed that a sprinkler system failure had occurred and therefore it was assumed a credible fire would be a full flashover on any one floor of the building.

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 modelled, 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 modelling purposes:

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

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.

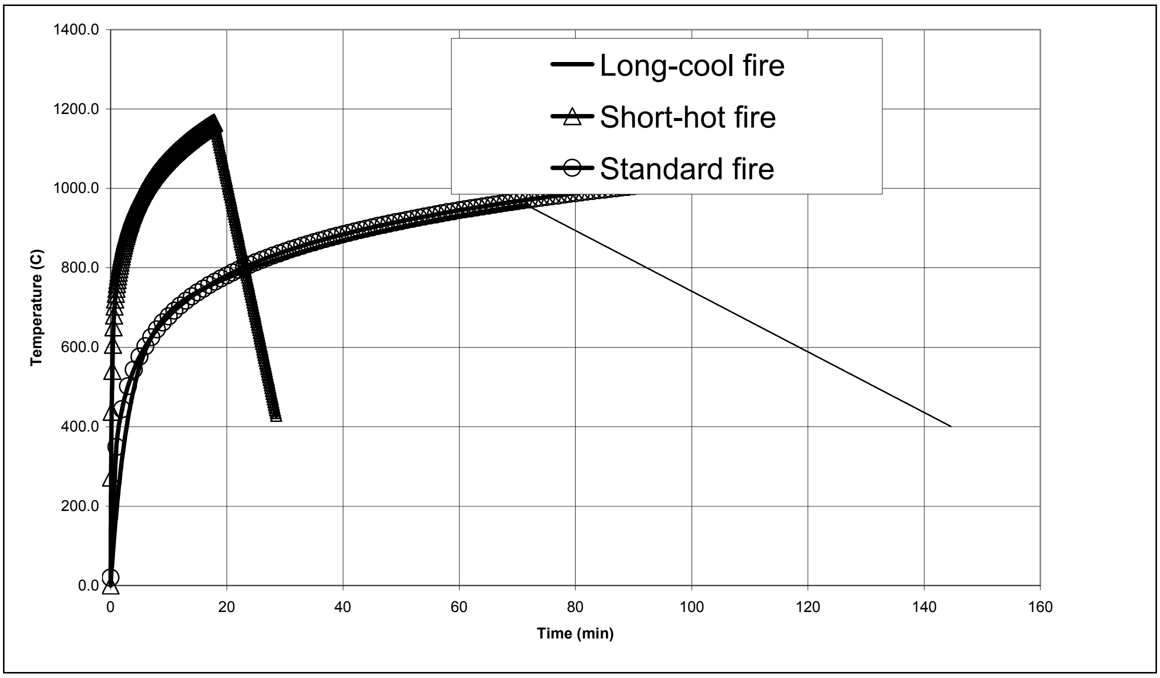


Figure 7 **Design fires for the structural analysis**

Steel temperatures as a result of the fire scenarios were calculated using the lumped mass heat transfer equations in Eurocode 3[9] 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 Figure 8-10 for each design fire.

A 1D finite element 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[11] and so is not repeated here.

The slab temperatures in structural models with the "design" fire exposures including cooling were modelled explicitly. The actual temperature gradients through the depth of the slab were modelled and are illustrated in Figures 11-12. 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.

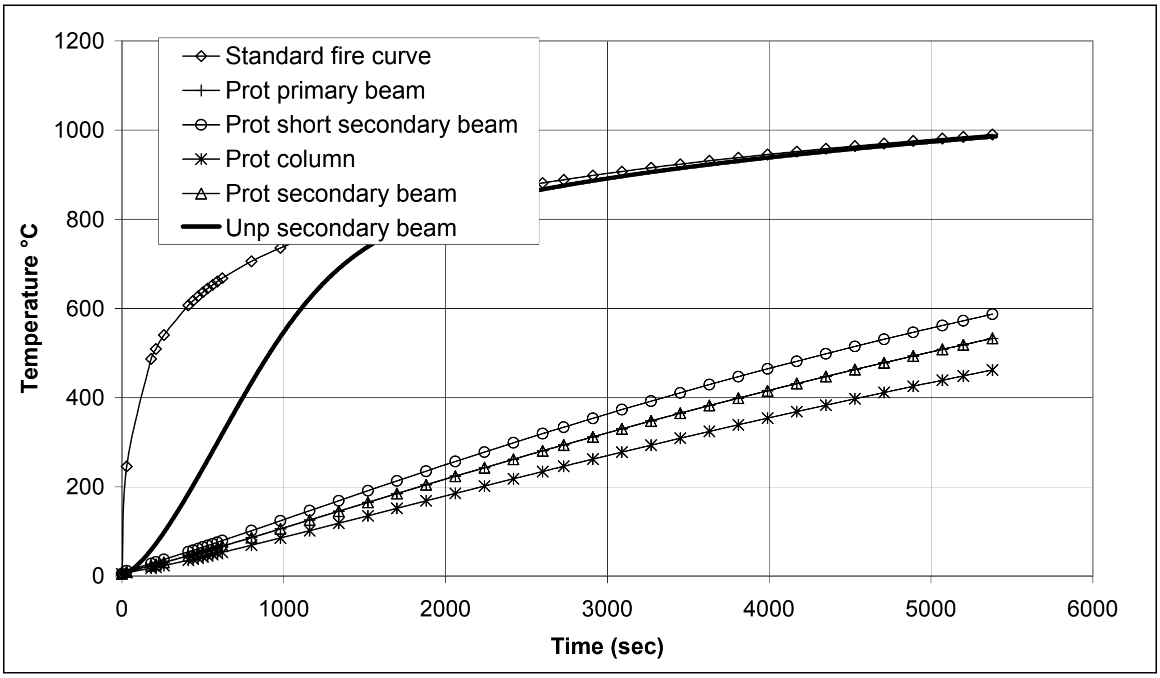


Figure 8 Steel temperatures used in the FE model with standard fire exposure (prot = protected, unp = unprotected).

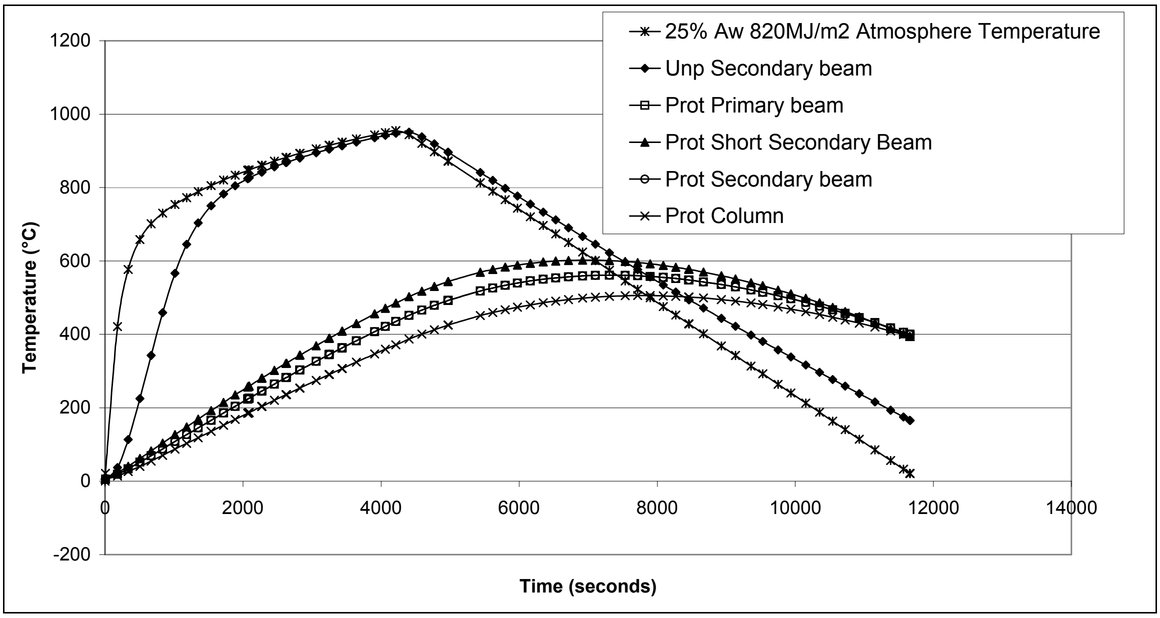


Figure 9 Temperature histories in the steelwork for the design fire with 25% of the available glazing on one floor having failed providing ventilation (prot = protected, unp = unprotected).

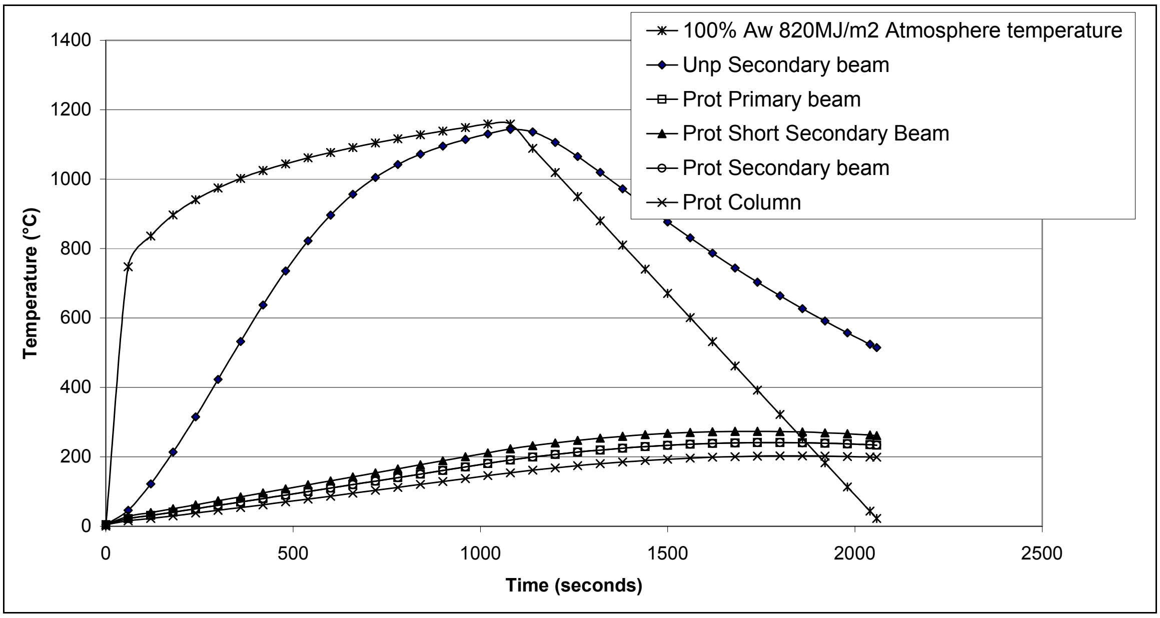


Figure 10 Temperature histories in the steelwork for the design fire with 100% of the available glazing on one floor having failed providing ventilation (prot = protected, unp = unprotected).

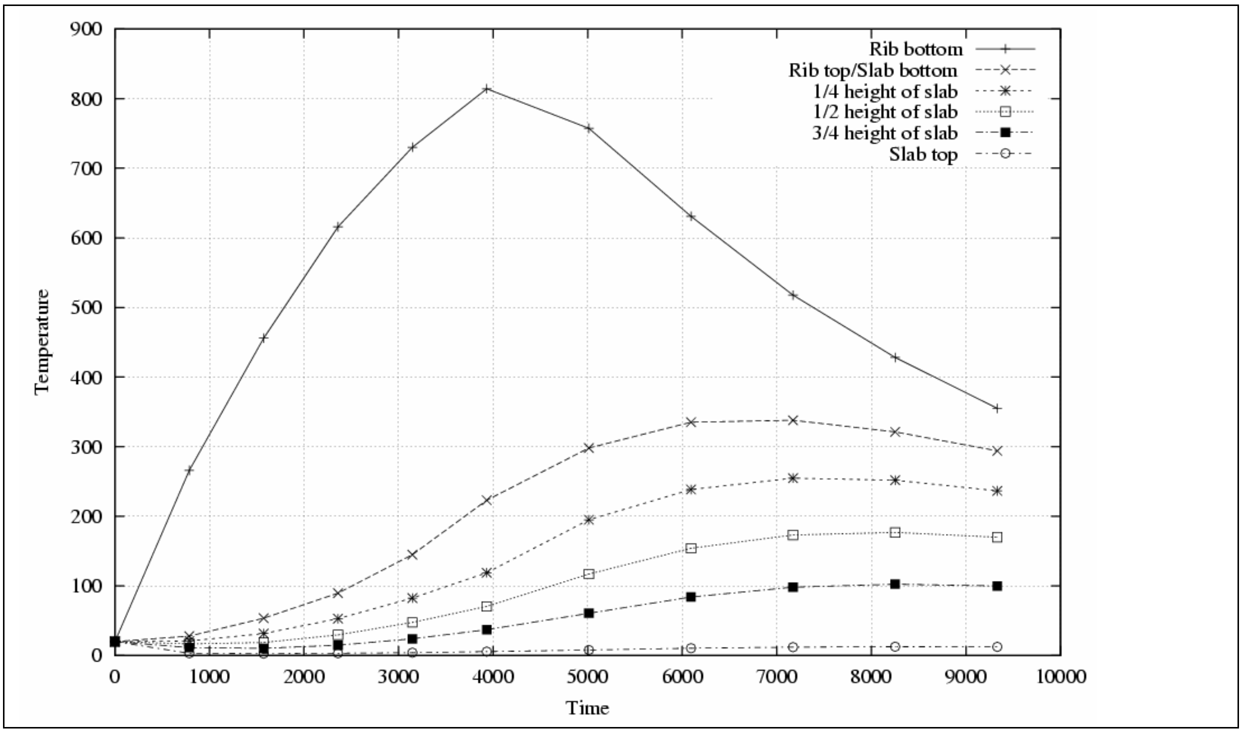


Figure 11 Temperature histories in the concrete slab for the design fire with 25% of the available glazing on one floor having failed providing ventilation

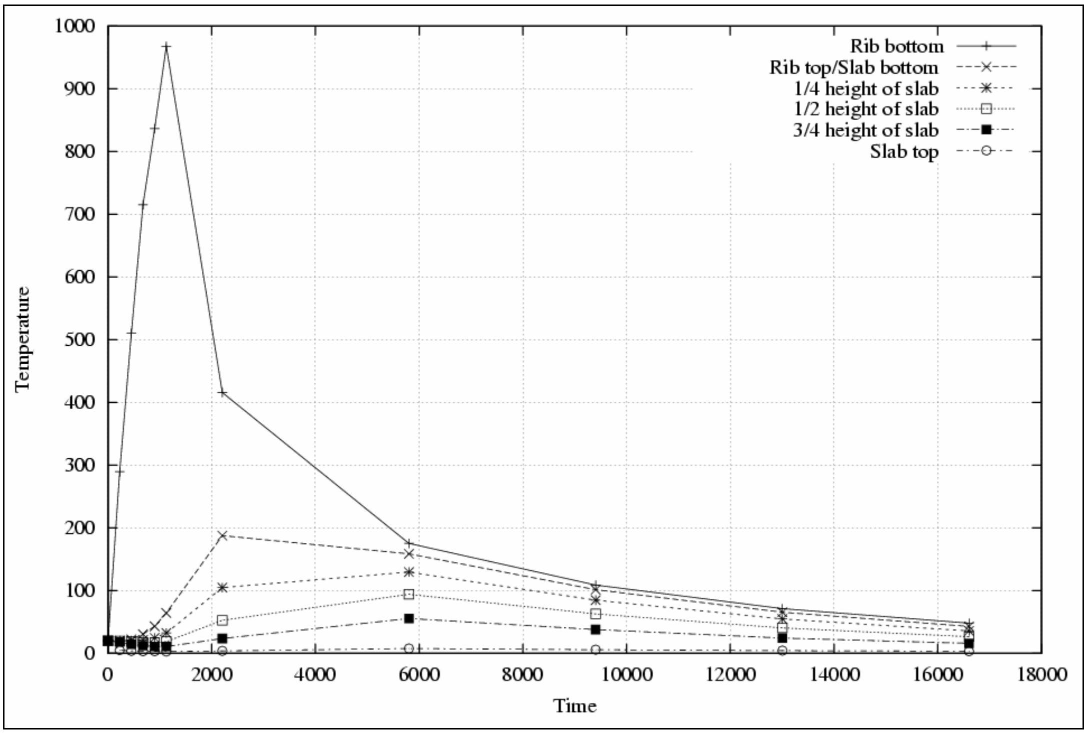


Figure 12 Temperature histories in the concrete slab for the design fire with 100% of the available glazing on one floor having failed providing ventilation

For each structural analysis it was assumed 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 they would be exposed to heating on all 4 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: GLOBAL MODEL 1 (9m 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 mid-span 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 mid-span 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.

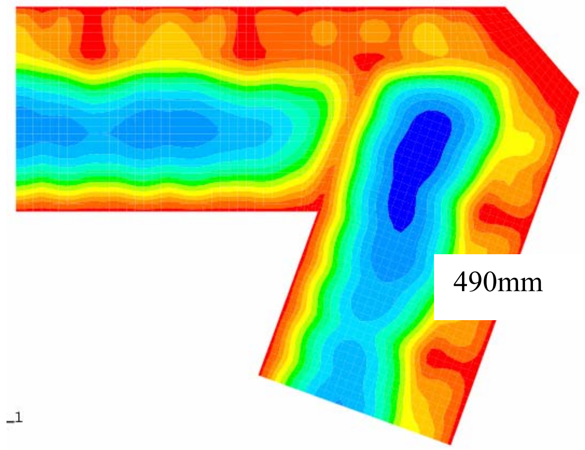


Figure 13 Contour plot of deflection at the end of heating

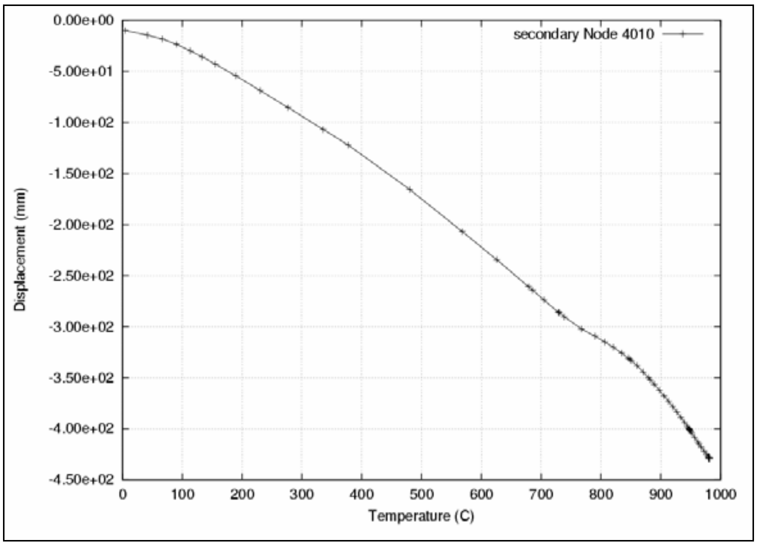


Figure 14 Mid-span deflection of a typical unprotected secondary beam (secondary node 4010 = node number at mid span of the secondary beam in the model).

The axial force at mid-span 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.

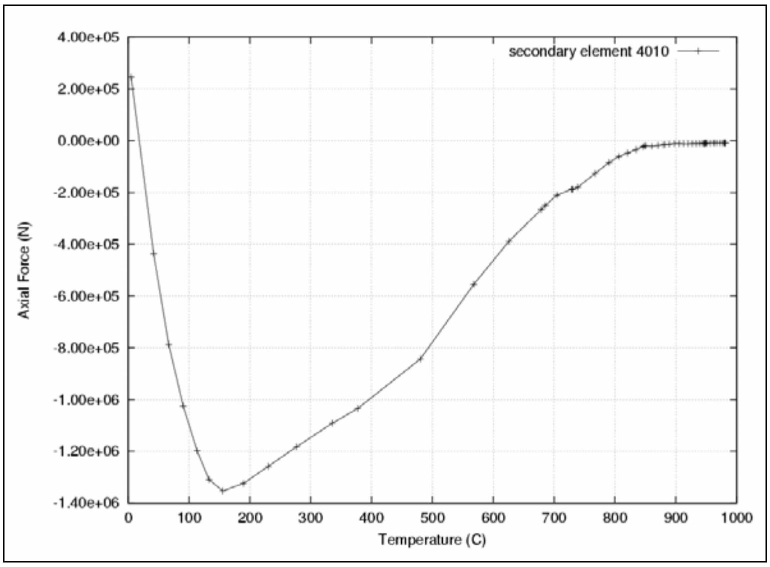


Figure 15 Axial force at mid-span of the unprotected secondary beam

The total strains (thermal + mechanical) in the slab at reinforcement level are plotted in Figure 16 for the Y(=2) direction. Compressions have 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 boundary. Any localised 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 at the fire limit state was tested by a detailed connection model briefly described later in this paper.

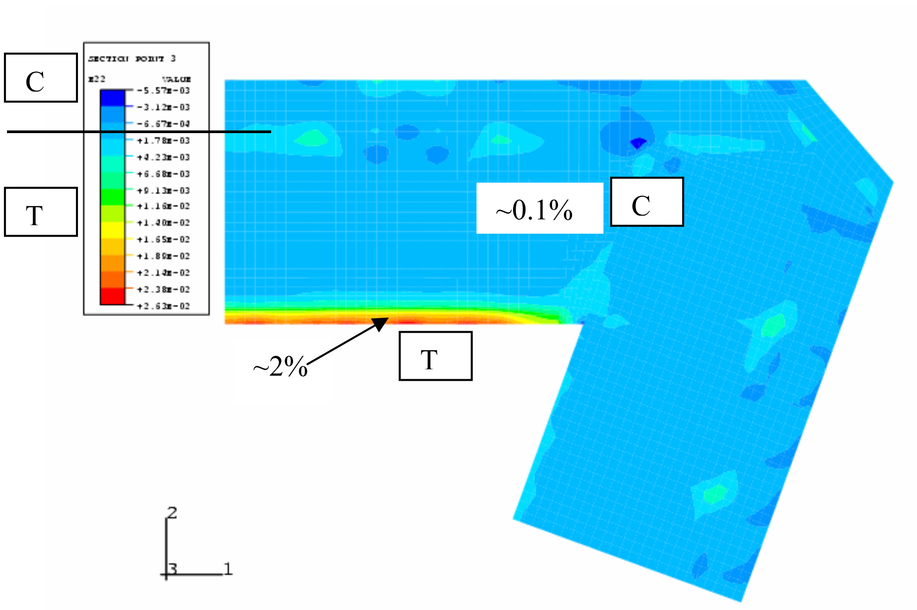


Figure 16 Strain in the 2 (Y) direction at the level of the reinforcement in the slab after 90 minutes of the standard fire exposure. C=compression, T=tension.

RESULTS: GLOBAL MODEL 1 (9m spans) Fully protected structure in response to the standard fire

During the design process a direct comparison was made between the structural behaviour observed in the ABAQUS model of the proposed design (reported in the previous section of this paper "GLOBAL MODEL 1 [9m spans] Proposed structure with unprotected secondary steel beams in response to the standard fire") 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 390mm. Most secondary steel beams deflect up to 200mm. This is contrary to the common belief that protected structure does not deflect. Note also it 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 490mm and the mid-span deflection of the unprotected secondary beams is about 450mm (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 results in identical 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 minutes. This is based on the UK requirements etc for fire proofing.

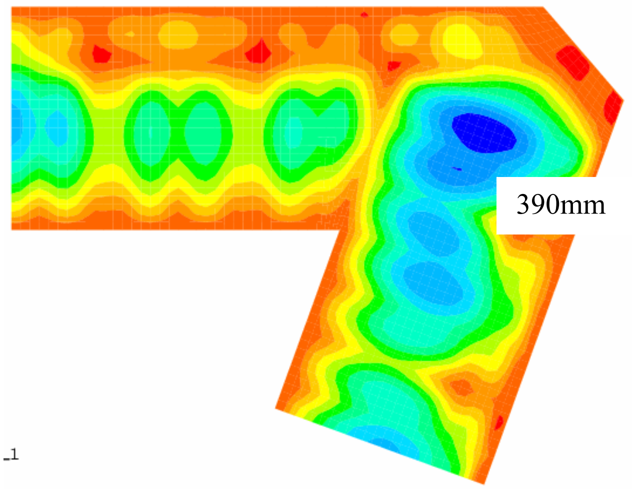


Figure 17 Contour plot of deflection at the end of heating

The deflection at mid-span of a typical fully protected secondary beam is very linear i.e. the rate of deflection is not changing (see Figure 18). The same behaviour 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 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.

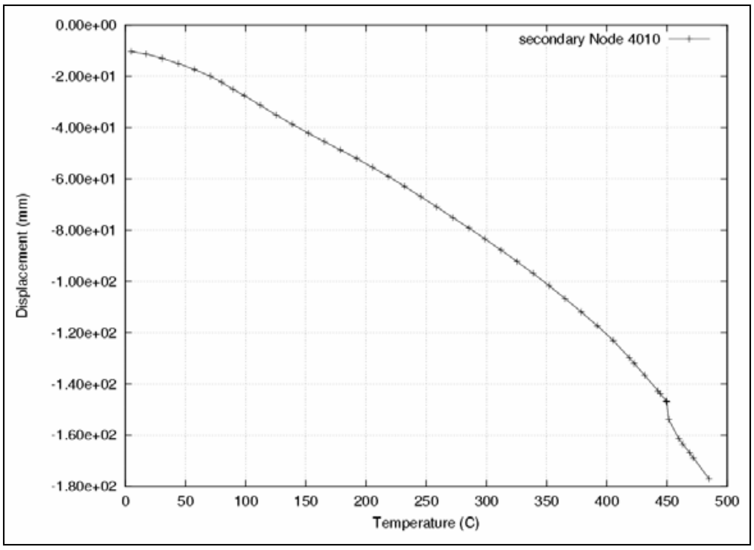


Figure 18 Deflection at mid-span of the secondary beam

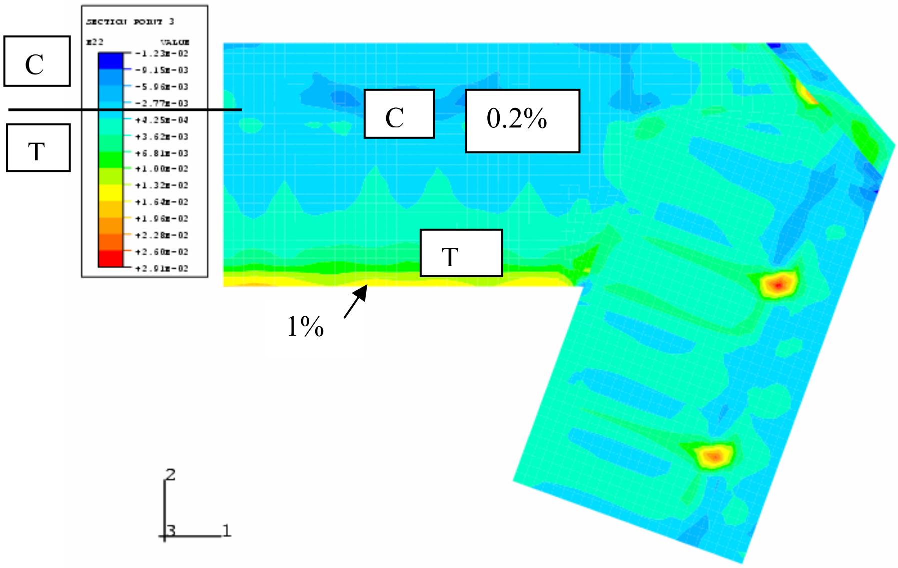


Figure 19 Strain in the 2 (Y) direction at the level of the reinforcement in the slab. C=compression, T=tension.

SUMMARY : SIGNIFICANCE OF RESULTS FROM GLOBAL MODEL 1 (9m spans) IN RESPONSE TO THE STANDARD FIRE

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 mid-span deflection of the floor slab was 390mm when all beams were protected and only ~100mm 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 finite element 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 behaviour is vastly different from the standard furnace test. The modelling 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 generalised conclusions cannot be made.

RESULTS-MODEL 2 (10m Spans)
Model 1 captured the most novel part of the structure at Mincing Lane including the masonry façade and the slab spanning in 2 different directions. However, the spans in this region are 9m whereas on the opposite side of the floor plate at the rear of the building beams span 10m. These larger spans generated greater thermal expansion and therefore greater mid-span deflections. Larger tensions were observed at the slab to core interface and runaway failure was observed. The remainder of this paper therefore discusses the differing response of this portion of the frame to the two parametric design fires and the design changes made to mitigate failure as a result of the observed results.

RESULTS-MODEL 2 (10m spans) with a (1) "short-hot" fire
Figure 20 shows the deflection at mid-span (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 heating due to the thermal lag concrete experiences when exposed to temperature. 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.

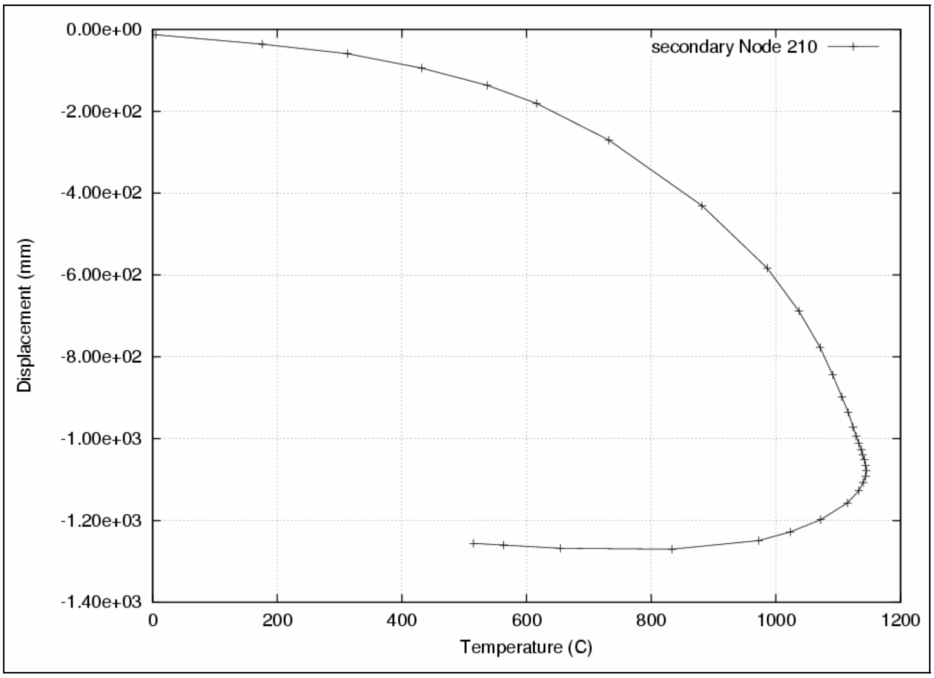


Figure 20 Deflection at mid-span of the secondary beam at the centre of the model

Figure 21 shows the axial force at mid-span 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 decreases with increasing temperature. Ultimate yield of the steel beam takes place at about 500°C.

Between 800°C 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 utilise 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 heating as a result of thermal lag) pushes the steel beam downward and the steel tries to contract into its original position.

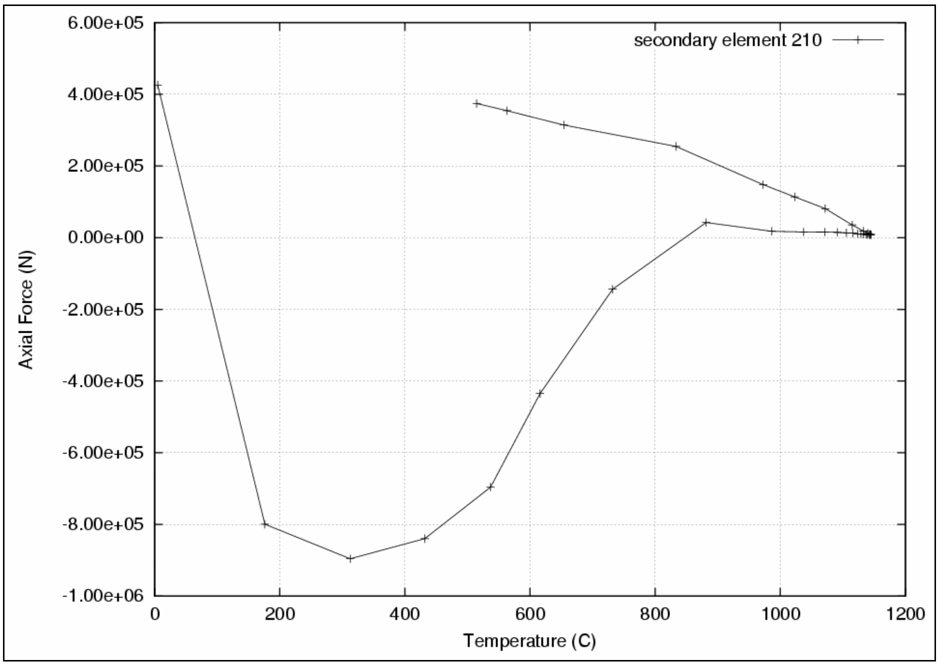


Figure 21 Axial force at mid-span of the secondary beam

A plot of strain at the level of the reinforcement in the slab in direction 2 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% whilst 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 and was addressed by protecting the secondary beams between the columns to reduce deflections and the analysis repeated. Note: 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.

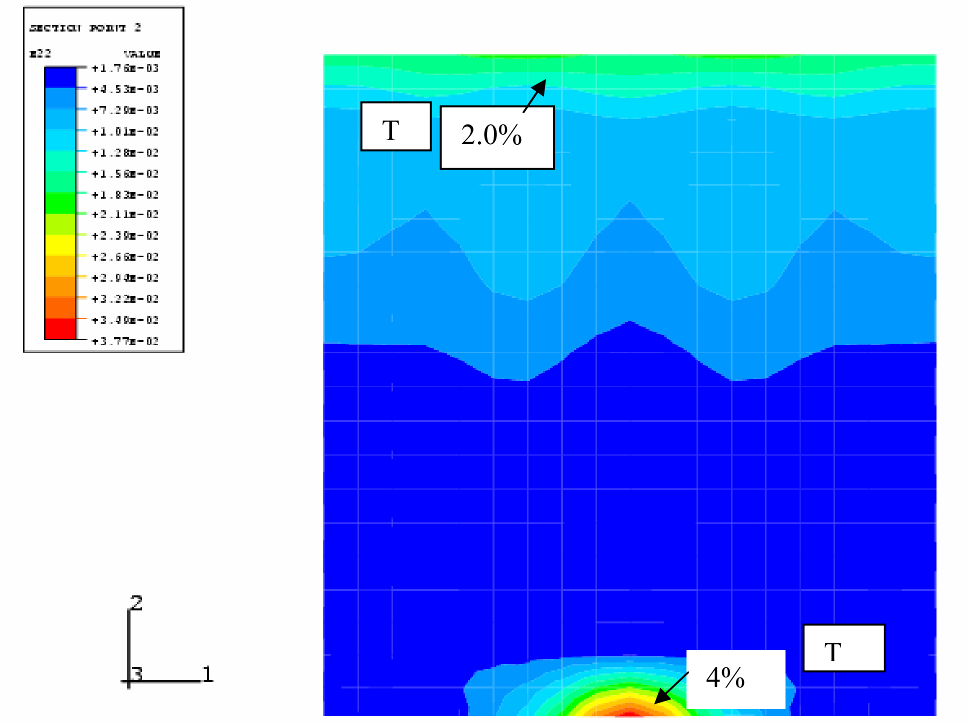


Figure 22 Strain in the 2 direction at the level of the reinforcement in the slab at the end of heating.

RESULTS – MODEL 2 (10m spans) -"short-hot" fire secondary beams protected between columns

The deflection at mid-span of an unprotected secondary beam is shown in Figure 3.5A 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 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.

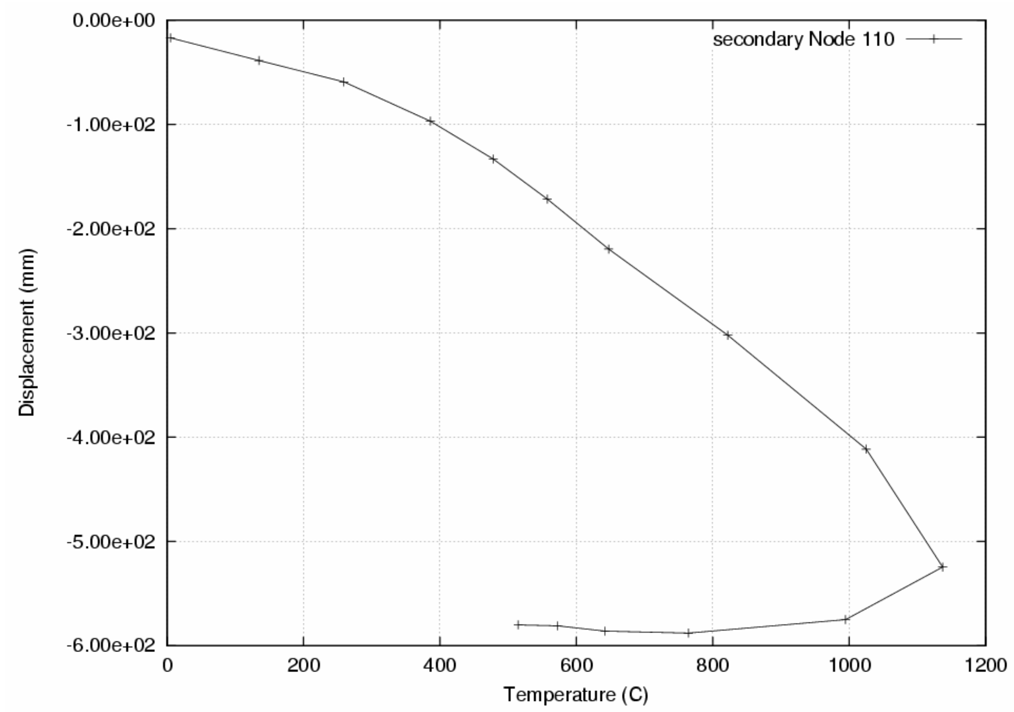


Figure 23 Deflection at the mid-span of the secondary beam

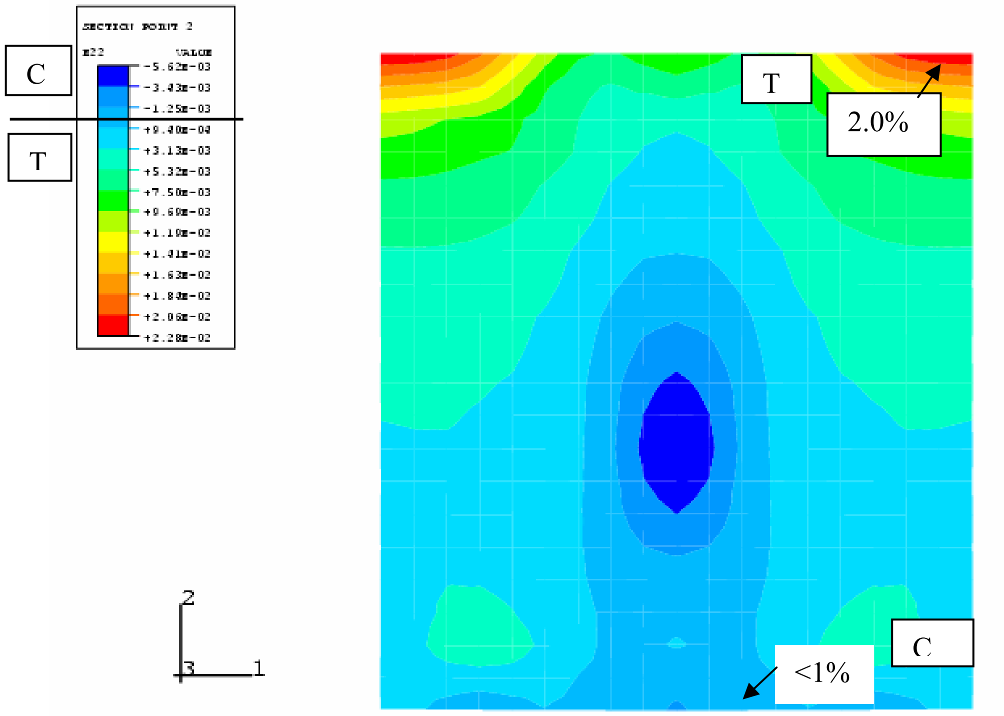


Figure 24 Strain in the 2 direction at the level of the reinforcement in the slab at the end of heating

RESULTS – MODEL 2 (10m spans) with a (1) "long-cool" fire

Mid-span 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 slab had been modelled the failure is unlikely to have been observed. However, as the conservative finite element model did show failure for the "long-cool" design fire the secondary beams between columns were protected and the analysis was repeated.

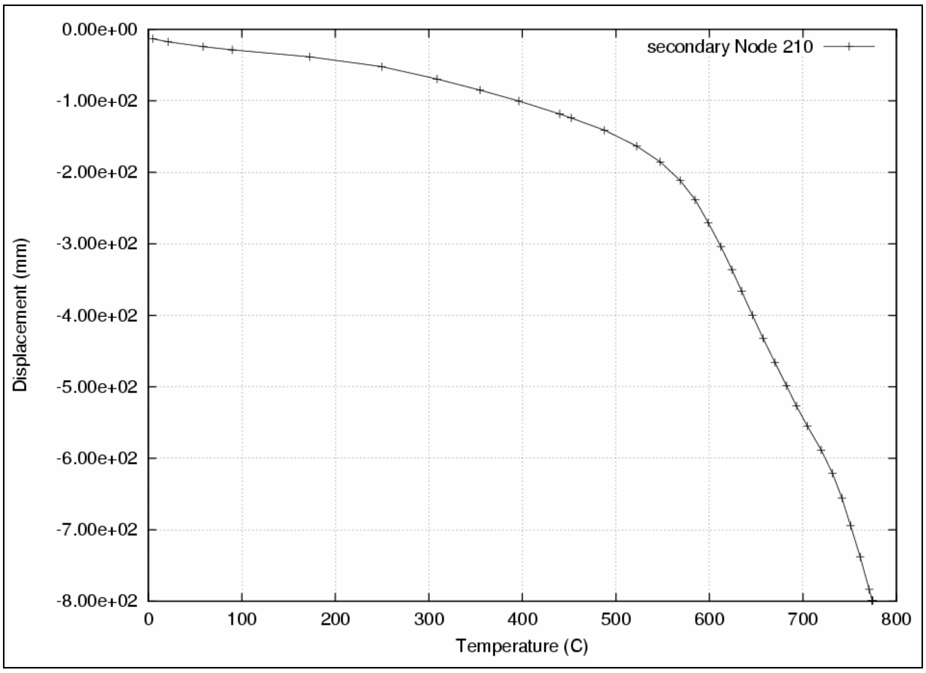


Figure 25 Runaway deflections of the secondary beam mid-way through the heating phase of the fire

RESULTS – MODEL 2 (10m spans) with a (1) "long-cool" fire and with (2) secondary beams protected between columns

The mid-span 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. Deflections are almost all associated with thermal effects. At unprotected steel temperatures of 800°C the protected steel beam will 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.

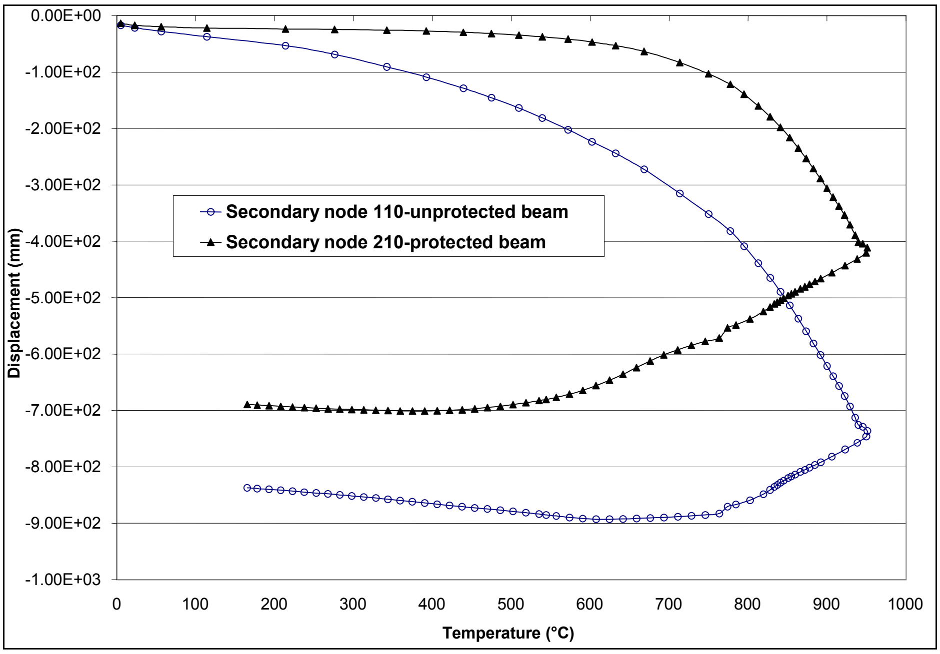


Figure 26 Mid-span deflection of a protected and unprotected beam in the "long-cool" fire (see Figure 5 for the location of nodes 110 and 210).

The strains at the level of the reinforcement are shown in Figure 27 at the end of heating in the 2 direction. Maximum total tensile strains of about 4% occur in regions along the core edge. Again this is high and is partly to do with 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 failure did not occur.

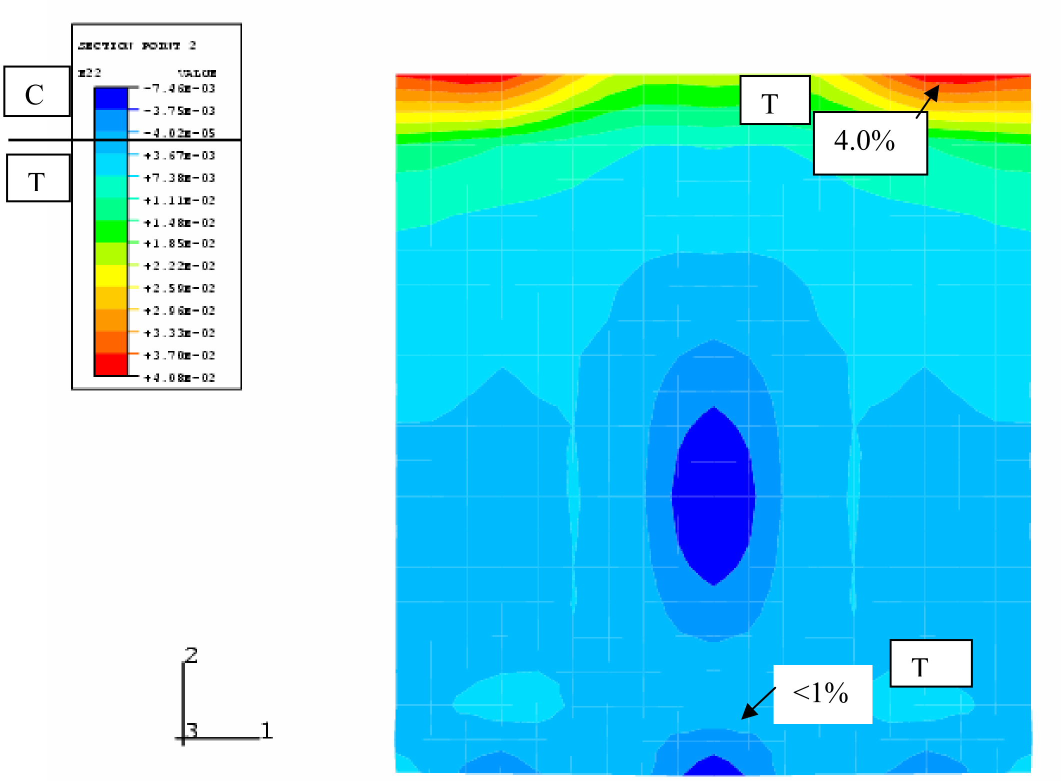


Figure 27 Strain in the 2 direction at the level of the reinforcement in the slab at the end of heating

SUMMARY: SIGNIFICANCE OF THE RESULTS FROM THE 10M SPAN GLOBAL MODEL (MODEL 2)

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 behaviour seen in the 10m 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 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 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 MODELLING AT THE FLOOR TO THE CORE BOUNDARY

The global finite element modelling of the composite floor slab showed a number of the secondary steel beams could be left unprotected as they are not essential to the stability of the floor slab in fire. However, the global FE model in ABAQUS 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 localised 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 credible design fire defined by the global FE analysis as the “long-cool” fire.

CONNECTION MODEL DESCRIPTION

The local model represented a 10m 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 5m x 4.5m 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 modelled as rigid and fixed. The 10m 305UC97 steel beams were modelled with non-linear 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 modelled as 130mm thick with a reinforcement grid of T6 bars at 200mm spacing (0.22% reinforcement). T10 bend-out bars at 300mm 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 10m unprotected beam consists of a plate cast-into the core wall with a fin plate and 3 M20 Grade 8.8 bolts connecting the fin plate to the web of the beam. The bolts are situated within 22 x 122mm 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 of the fire protection modelled covered the cast-in plate, the fin plate and the end 225mm of the secondary beam flange and web. All of the material properties and temperature time-histories were the same as those in the ABAQUS global models for the "long-cool" fire. The connection temperature was assumed to equal the protected secondary beam temperature.

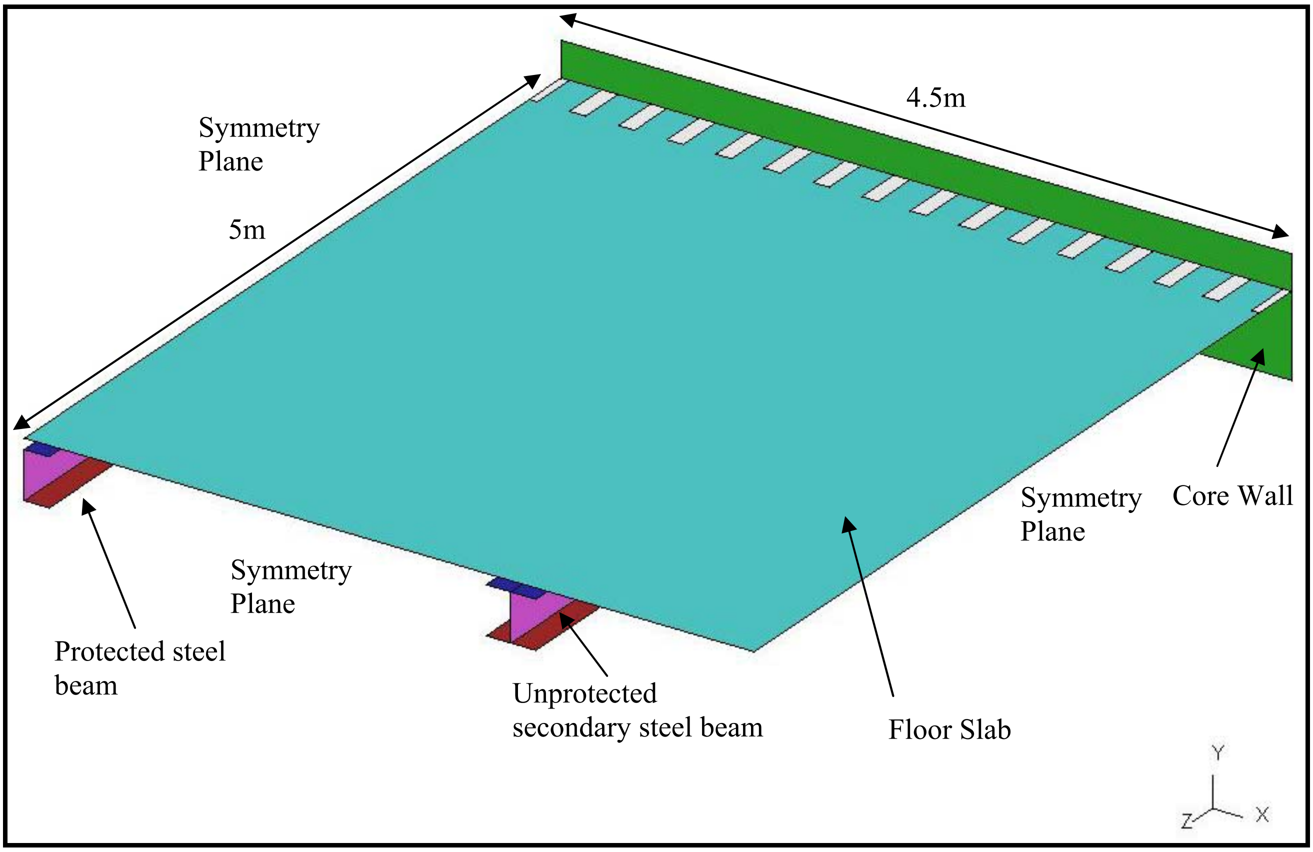


Figure 28 The LS DYNA model

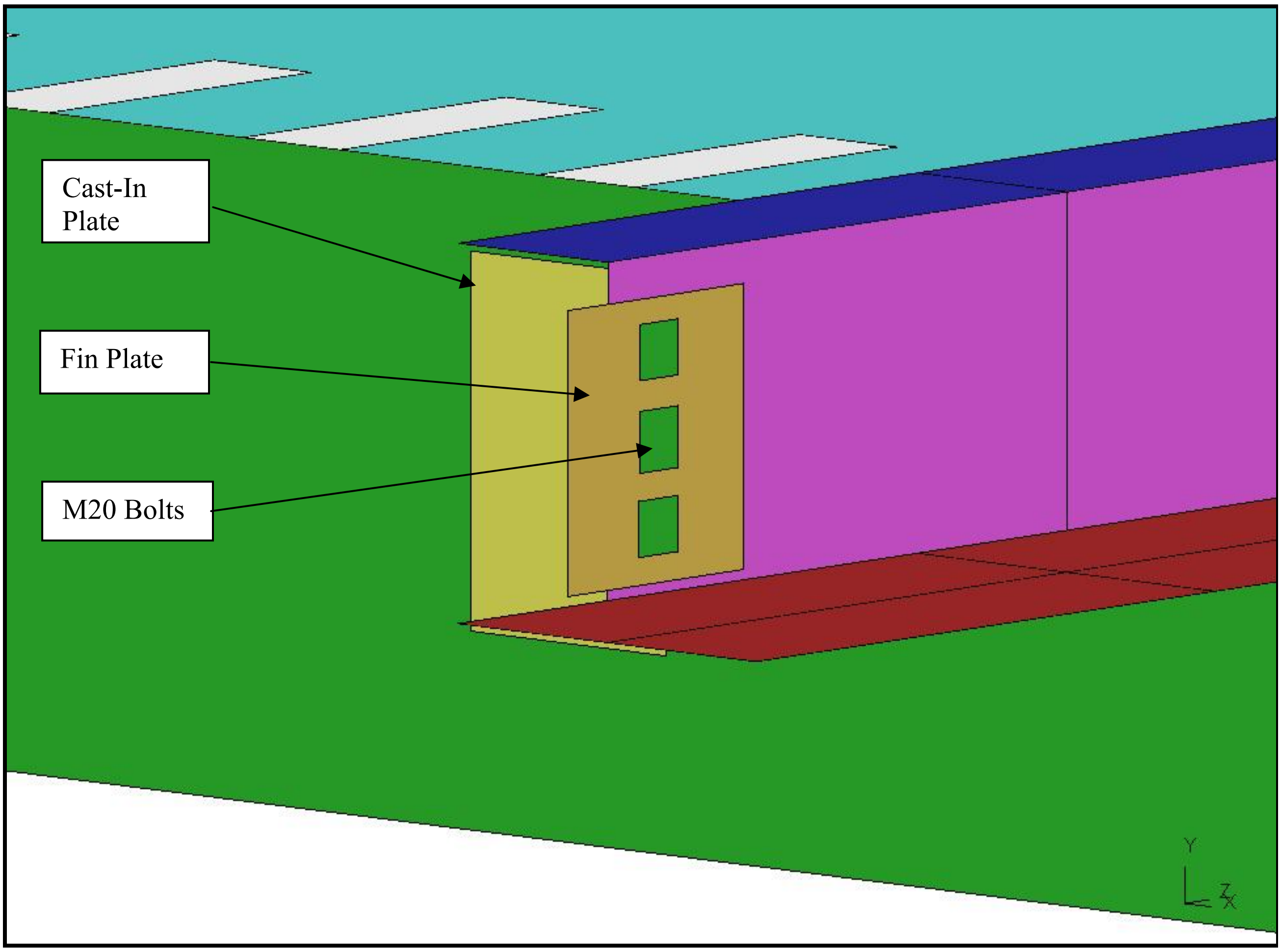


Figure 29 The connection detail in the LS DYNA model

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 downwards 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°C 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 come 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 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.

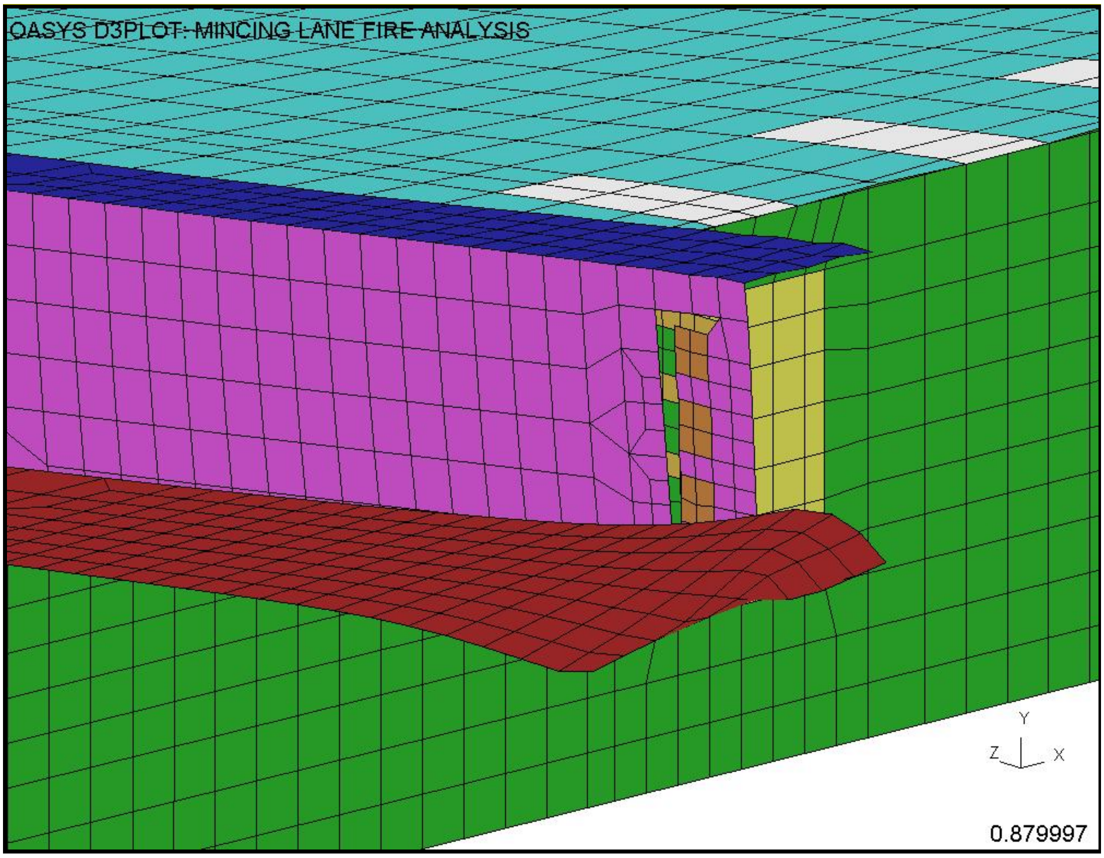


Figure 30 Web fracture around the bolts

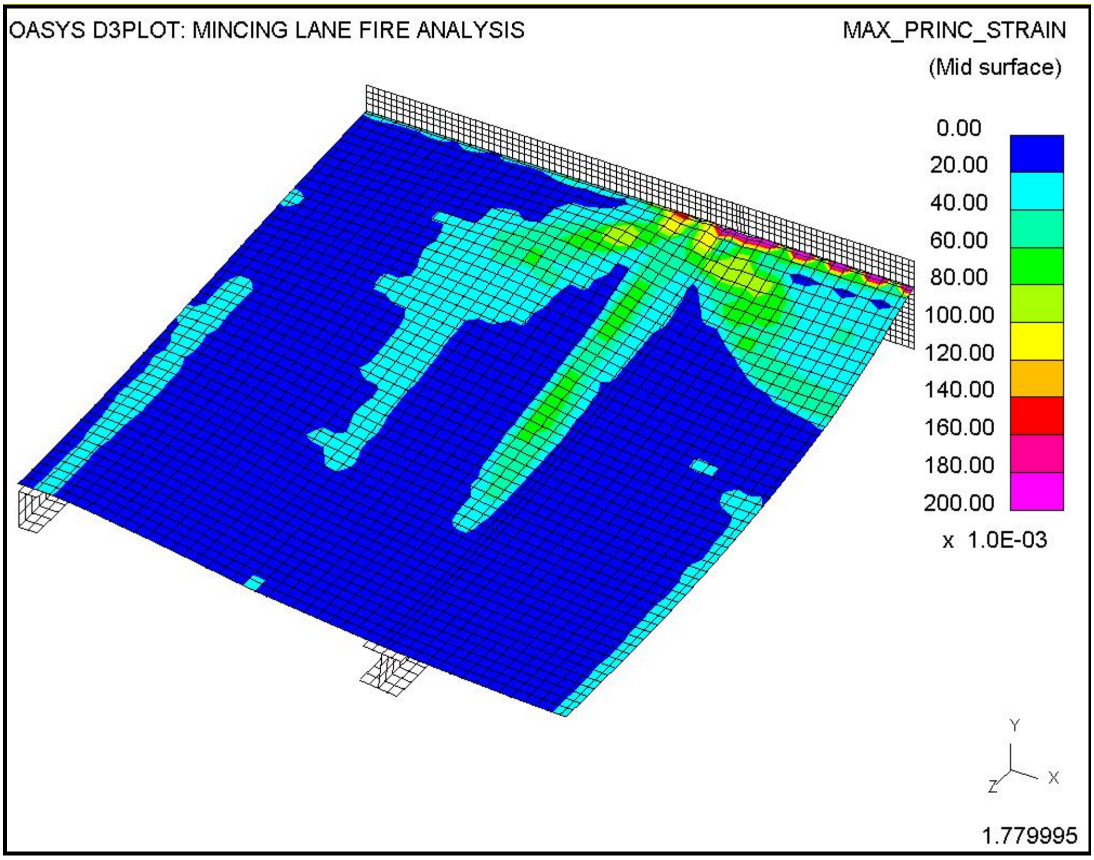


Figure 31 Principal strains

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.

CONCLUSION

This paper provides a snapshot of information and analysis to demonstrate 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 the structural frame at Mincing lane, London if it were designed prescriptively with its performance in response to credible design fires and reduced passive fire protection all as part of a research approach used to satisfy the stakeholders of a structural fire engineering solution.

A detailed finite element analysis 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 behaviour of the proposed design case with secondary steel left unprotected and the structural response if all steel had been fire protected as would be the case in a traditional prescriptive design. The results of the fully protected model were similar to those from the proposed design case 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 design they would normally approve, where it complies with prescriptive guidance, and the performance of the proposed design with some bare steel.

A conservative model of the 10m span bays in the Mincing Lane 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 paper are applicable to this building design only.

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