

THE INFLUENCE OF CONNECTIONS ON THE ROBUSTNESS OF COMPOSITE STRUCTURES IN FIRE

I. W. Burgess

Department of Civil and Structural Engineering, University of Sheffield,
Sheffield, S1 3JD, UK

ABSTRACT

Connections are key components in framed structures, tying floor beams either directly to supporting columns or to primary beams. Failure of a connection could cause collapse of the connected beam, which could lead to further buckling of the column as it loses its lateral support, or to progressive collapse of floor slabs. In performance-based fire design, beams are sometimes allowed both to develop very large deflections and to support the floor by developing large catenary tension forces. The tying capacity of connections to transfer these significant forces to adjacent structure when they are already subject to high rotations is essential to maintain structural stability. The development of tying forces in connections within steel-framed buildings with composite floors, at various stages of fire, is described, as the background to an experimental study of connection behaviour under combined forces. Traditionally, connections have been studied in terms of their moment-rotation behaviour alone, although some research has been performed on the tying capacity of connections at ambient temperature, mainly conducted in terms of a purely normal tying force. However, a number of tests have now been carried out in a project which investigated the behaviour of commonly used steel connections subjected to an inclined tying force at elevated temperatures. The effect of co-existent shear force and rotation on the tying capacity was studied in furnace tests at different temperatures. The paper discusses the results of these tests on four different connection types, and their implications. A key objective is to facilitate the inclusion of the major aspects of joint behaviour into global structural analysis for performance-based structural fire engineering design of steel-framed and composite structures by means of a component approach. The test results have been used to validate models of the principal components of such connections.

KEY WORDS

Steel and Composite structures, Fire, Robustness, Connections, Component-based modelling.

INTRODUCTION

The collapse of the twin towers of the World Trade Center in New York in 2001 (NIST 2005), after they had withstood massive structural damage caused by aircraft impacts, was a reminder of the potential of fire to cause devastating failures of high-rise buildings by initiating progressive collapse. In the UK, North America and increasingly in the Far East, steel-framed composite structures are very widely used for multi-storey buildings, because they are ideally suited to the current drive for improved construction efficiency as labour costs increase. A programme of fire tests on a full-scale medium-rise composite building at Cardington in the mid 1990s (British Steel 1999) showed very effectively that composite floors within a structural frame can resist the structural effects of fire significantly more effectively than is suggested by conventional tests on isolated elements. So far, however, neither prescriptive design to protection manufacturers' specifications (ASFP 2007), simple performance-based design to BS 5950 Part 8 (BSI 2000) or Eurocode 4 Part1-2 (CEN 2005b) nor more advanced design methods have provided any proven safeguards against progressive collapse.

All common building materials lose strength when heated to high temperatures. For steel the change can be seen in the stress-strain curves at temperatures as low as 300°C. Although steel does not melt until about 1500°C, only 23% of the ambient-temperature strength remains at 700°C. At 800°C this has reduced to 11% and at 900°C to 6%. Concrete also loses strength as its temperature increases, because of a combination of internal cracking and chemical changes. As a brittle material the stress-strain curves for concrete are different in form from those for steel. The curves all have a maximum compressive strength, followed by a rapidly descending branch, and little tensile strength. Concrete also differs from steel in that it does not recover strength on being cooled from high temperatures, which is significant to refurbishment of buildings after fires.

The most significant additional effect caused to materials by temperature change is thermal straining, which changes the natural dimensions of components made of a single material. Within the typical ambient temperature range the coefficients of thermal expansion of low-carbon structural steels and concrete are quite similar, which is an important property in making reinforced concrete a viable structural composite for normal conditions. When the natural dimensions of parts of a composite member change markedly this can cause very significant residual stresses, as well as overall distortion of the composite member, in achieving equilibrium. The thermal expansion coefficient of structural steel stays reasonably constant until its crystal structure begins to undergo a change between 700-800°C, whereas that of normal-weight concrete increases by a factor of 4 from the ambient range to about 700°C, beyond which temperature its chemical structure breaks down considerably. It is therefore clear that interaction between steel and concrete because of high temperatures and temperature differentials can be a major influence on the structural behaviour in fire.

Fire resistance requirements

Fire safety requirements for buildings are set by law, and attempt to reflect the risks that fires pose to the occupants and to fire-fighters. The general fire safety scenario for a building includes provision of adequate means of escape for occupants, fire detection and control, together with a fire resistance requirement for the structure which is expressed as a period of time. It is not generally appreciated by structural engineers and architects that these are not guaranteed periods of safety if a fire occurs, but actually relate to various fairly arbitrary tests of structural behaviour (mainly limiting deflections) in a totally artificial fire atmosphere temperature-growth environment which is recognized internationally as the ISO834 (ISO 1985) Standard Heating Curve (Figure 1) and is included in Eurocode 1 Part 1-2 (CEN 2002). This time-temperature curve has no correlation with the actual fire load, ventilation conditions or compartment characteristics which apply in a particular design case, and includes no

peak temperature or decay phase. This curve may be assumed to start at flash-over, and its temperature then increases indefinitely, making it completely unrealistic as a model of a real fire.

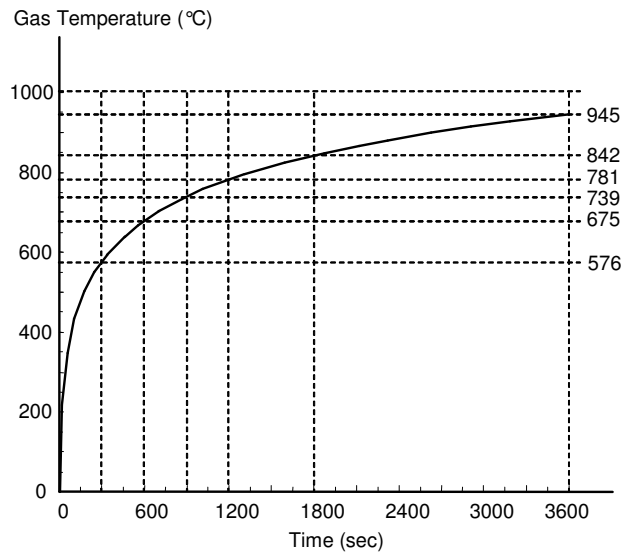


Figure 1: ISO 834 Standard Fire curve.

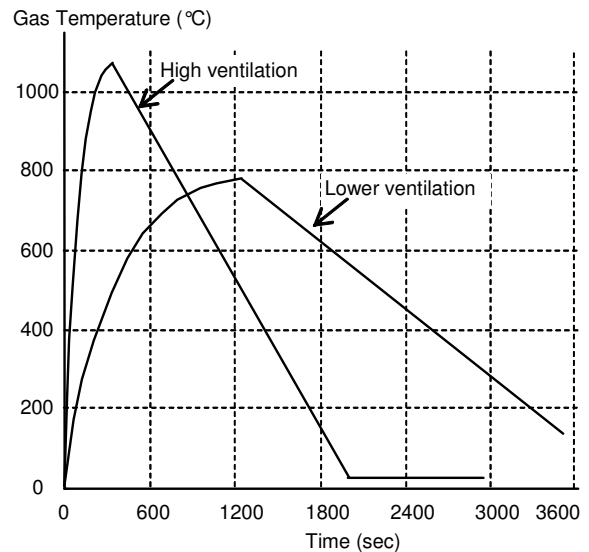


Figure 2: Eurocode 1 Parametric fire curves.

The advent of the Eurocodes has introduced a significant opportunity to treat the fire itself in a much more realistic way than has been possible in the context of fire resistance periods based on the Standard Fire time-temperature curve (CEN 2002). A calculation method now exists, also in Eurocode 1 Part 1-2 (CEN 2002), which enables a reasonably realistic time-temperature representation of the post-flashover phase of a natural compartment fire, including the peak temperature and the cooling phase. This is known as the Parametric Fire, illustrated in Figure 2 for two different degrees of ventilation, and needs data about the compartment dimensions and wall materials, the fire load density and the ventilation conditions. If it is possible to calculate critical temperatures, either for individual elements or of an integrated zone of the structure, then these can be compared against the actual structure temperatures developed in the same way as is done for the Standard Fire. However, instead of checking whether a critical deflection has been exceeded in the required fire resistance time the structure can be checked for stability at all times until it has completely cooled. This is effectively an infinite fire resistance based on complete “burn-out” of the compartment.

METHODS OF ESTABLISHING FIRE RESISTANCE

Prescriptive fire protection

Given the 100 years of accumulated experience of using the Standard Fire as a basis, its use as a thermal context can be justified logically in rating the effectiveness in insulation and integrity of separating elements such as walls, floors and doors. It may be argued that the same logic applies to the load-carrying capacity of standard, isolated structural elements such as beams, columns and slabs, especially where these are connected in a simple hierarchical sequence in which each element simply accumulates vertical load and transfers this as statically determined reaction forces to an element of higher-order in the hierarchy. This is justifiable for simple structural systems such as traditional gravity-frame steel construction in which slabs are composed of one-way spanning pre-cast concrete planks supported as single spans on primary beams whose joints to columns have no significant rotational stiffness. Under such conditions each member carries well-defined internal forces which do

not change because of interactions with the adjacent structure. The most usual way of achieving the fire resistance requirement for a steel-framed structure is simply to cover the exposed steel with a prescribed thickness of an insulating material, such as fire-resistant boarding, cementitious sprays or intumescent paints which expand to form a thick lightweight coating when heated above about 300°C. The required thickness of any of these systems has traditionally been prescribed by the protection manufacturers, with the sole objective of keeping the steel temperature below a fixed critical temperature (usually 550°C, or 620°C if supporting a concrete slab on the top flange) within the fire resistance period, irrespective of the loading level. This fixed critical temperature was originally based on reduction of steel yield strength until the normal ambient-temperature design safety factor of about 0.6, which was embodied in now-superseded permissible-stress design codes, had been fully eroded.

Single-member performance-based methods

A more rational performance-based approach, in line with the principles of limit-states design, was introduced in the UK in BS 5950 Part 8 (BSI 2000), and a similar method has more recently been included in Eurocode 3 Part 1-2 (CEN 2005a). This approach relates the critical temperature of any member to its load level at the Fire Limit State. This is logical because a lightly loaded member does not fail until it reaches a very high temperature, and will therefore survive much longer than a more highly loaded member. Codes of practice now accept that appropriate partial safety factors applied to characteristic loadings for the accidental limit state of fire are considerably lower than those used in ULS design for ambient temperature, and in consequence load ratios tend to lie well below 0.6, especially where variable loads exceed permanent loads. Hence the performance-based approach generally leads to lower requirements for fire protection than the prescriptive approach, and is clearly more realistic in at least this sense.

Tensile membrane action in composite floors

The Cardington experimental frame

Because of a widespread feeling that steel-framed structures are actually much more fire-resistant than simple design methods allow, a large experimental project was run in Britain (British Steel 1999) in the mid-1990s. A full-scale eight-storey building, typical of modern medium-rise office structures (but without architectural finishes or building services), was constructed inside a massive hangar at Cardington which had originally been built to house airships in the 1920s. All the floors were loaded with sandbags to a level typical for offices in the UK. Six fire tests were performed on the building, with several hundred measurements of temperatures, deflections and strains being taken during and after the fires. All columns were insulated, because of the severe consequences of a column failure, but all the internal steel beams were left completely unprotected. Some of the fires were lengthy and severe; the temperatures of steel beams usually reached over 800°C, and in one test 1150°C, at which the steel retains about 2% of its design strength. However, despite the fact that these temperatures were well above the critical temperatures given by the design procedures described above, and the distortions of floors were quite high, no failures occurred in any of the tests. After the tests finished in 1996 the data produced stimulated an upsurge in research aimed at understanding the complex interactions that take place during a building fire and producing reliable numerical models to predict structural performance in fire. A typical floor plan is shown in Figure 3, showing also the location of one of the tests (the British Steel Corner Bay Test), which illustrates well the load-carrying mechanism at work at high temperatures. The test was carried out in July 1995 on a corner bay of the structure in a compartment 9.98m wide by 7.57m deep. The fire compartment was constructed using lightweight concrete blockwork, the top of which was detached from the slab soffit to allow free deflection of the structure above, with the gap being filled by a flexible insulating blanket.

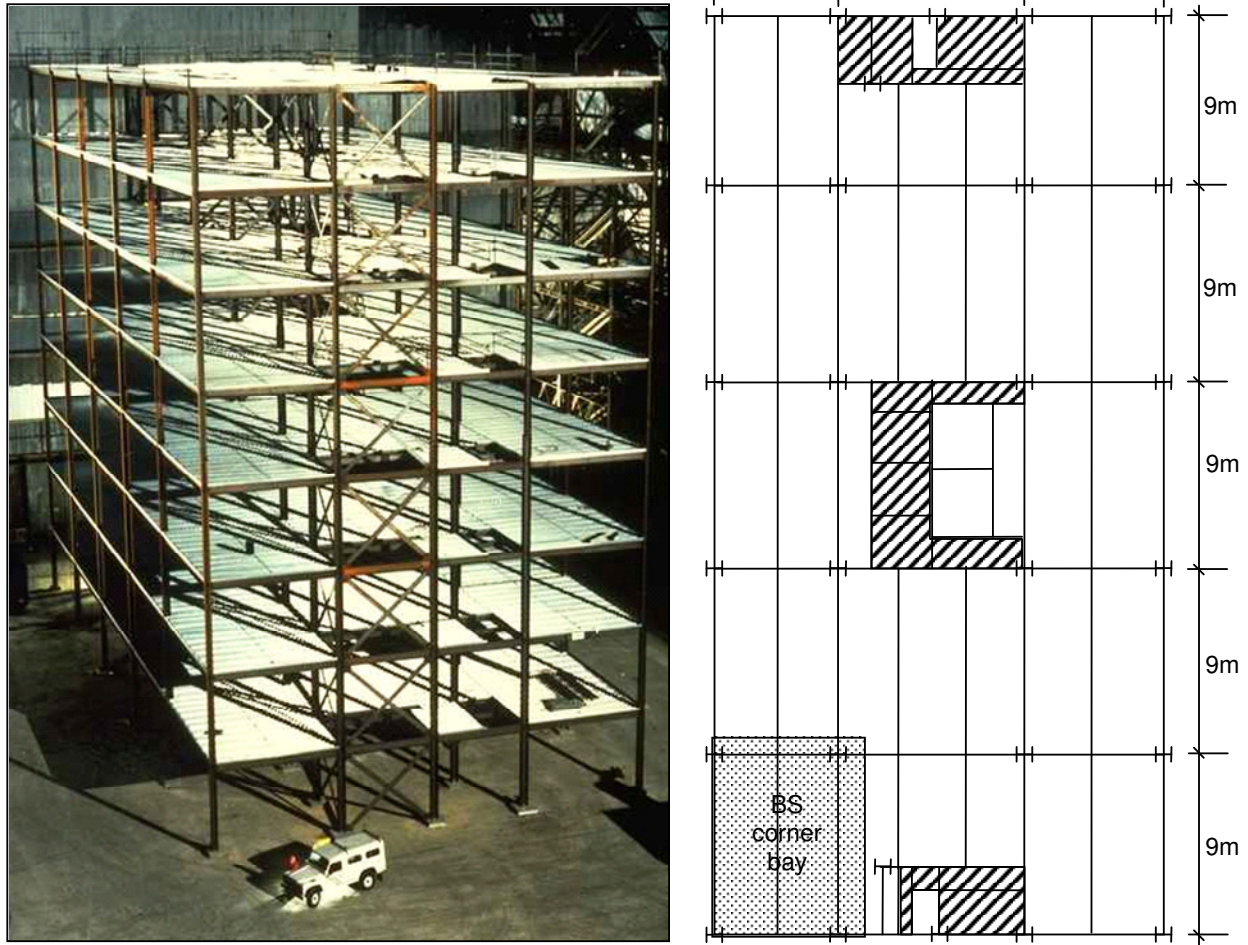


Figure 3: The Cardington test frame building, including typical floor layout and BS corner test location.

All columns and the two perimeter beams were wrapped with ceramic fibre insulation, but all other structural elements were left unprotected. The test was fired using timber cribs, giving an overall fire load of 45kg/m^2 , to produce a natural fire. During the fire test the maximum recorded atmosphere temperature in the compartment was 1028°C . The key observation from this test was that, although quite high deflections were achieved by the slab, no failure of either structural stability or integrity occurred during the test. This is despite the fact the temperature of the unprotected steel beams was well above 900°C . Existing prescriptive design would have assumed a critical temperature of 620°C because of the supported concrete slab. The more advanced design codes, BS5950 Part 8 (BSI 2000), and Eurocode 4 Part 1-2 (CEN 2005b), would have given a critical temperature for the secondary composite beams of about 670°C , based on their load ratio. This is especially surprising because steel at 900°C retains only about 6% of its normal strength, as compared with about 40% at 620°C and 25% at 670°C . The steel downstands of the composite beams clearly made only a minimal contribution to the survival of the floor slabs, and it was clear that the slabs themselves, at high deflection, had carried loads far in excess of those predicted by the normal design procedures. This observation caused previous research work on high-deflection behaviour of concrete slabs to be re-examined in the context of fire resistance. Although fairly large displacements had taken place, and these could be anticipated in any such situation, these are acceptable during a fire if it remains contained within the compartment of origin so that the risk of further fire spread is low.

The load-carrying behaviour of floor slabs

The behaviour of floor slabs in multi-storey frames in which the floor slabs simply impose load on steel beams (Figure 4) may be quite different from that of composite frames in which the slabs and steel beams are continuously connected.

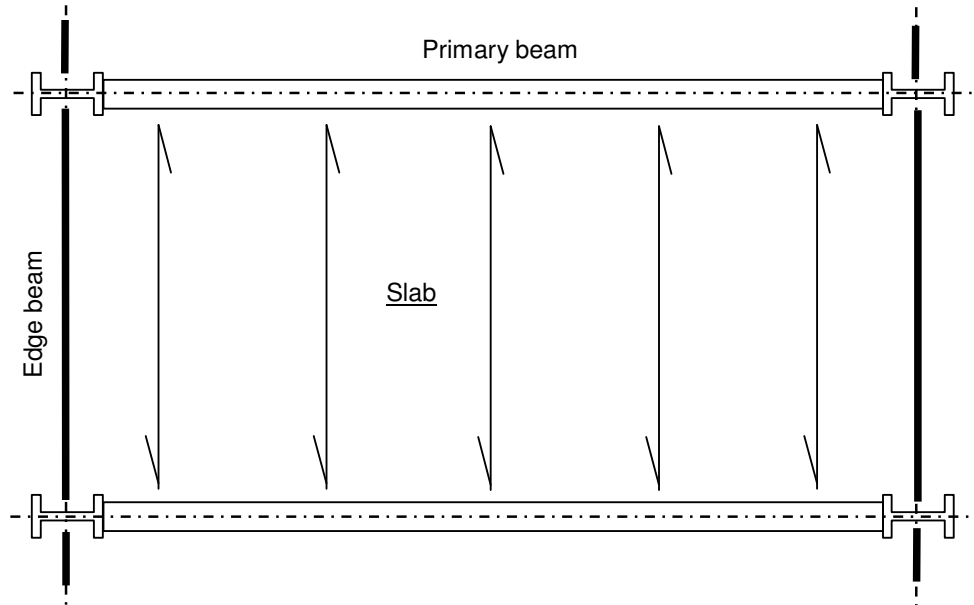


Figure 4: Typical non-composite floor framing system; load path at ambient temperature.

Free-standing slabs need only to be designed for local strength in fire, and do not change the structural action of their supporting beams, in contrast to floor slabs of composite construction (Figure 5). Both types of framework obviously depend to a large extent on the continued integrity of the joints between members, but the changes in joint forces are much more marked when there is the degree of continuity between heated and cool areas which is afforded by composite flooring systems.

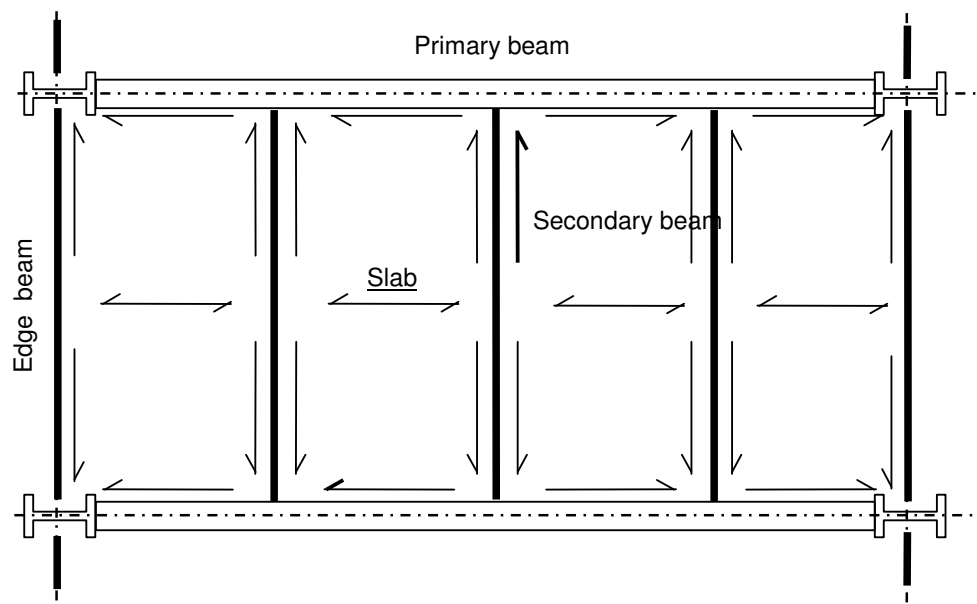


Figure 5: Typical composite floor framing system; load path at ambient temperature.

It is clear that a composite floor system like that shown in Figure 5 typically passes through different phases of behaviour as temperatures increase in a compartment heated by an intense, long-lived fire.

- Initially the exposed steel beams heat rapidly and expand, with little reduction in strength.
- The concrete slab heats more slowly, causing thermal bowing towards the heat source.
- Progressive reduction in steel strength and stiffness then causes very high permanent compressive strains in the steel beams.
- Restraint to thermal expansion, caused by the cool structure surrounding the fire compartment, further increases this compressive straining.

As the steel's strength reduces so the concrete slab to which it is attached plays an increasingly important role in supporting the floor loads. The characteristics of the slab, together with the way it is supported, will now control the way in which it carries its loads:

- The slab's residual flexural strength *may* at this stage be great enough for it effectively to carry the load unaided at relatively low deflections, especially if it has been designed with structural reinforcement rather than a nominal anti-cracking mesh, or if it is thicker than normal (say to control noise propagation in residential buildings).

If the slab is well supported against vertical deflection along lines which divide it into reasonably square areas, for example by the primary and secondary beams on the column gridlines, then tensile membrane action (Figure 6) can be generated as a load-carrying mechanism. The slab is then forced into double-curvature and hangs as a tensile membrane (although with the rigidity in in-surface shear that is more characteristic of a shallow shell) in its middle regions, while a peripheral compressive "ring-beam" is generated either around its supported periphery or in its edge-beams. This forms a self-equilibrating mechanism which supports the slab loading.

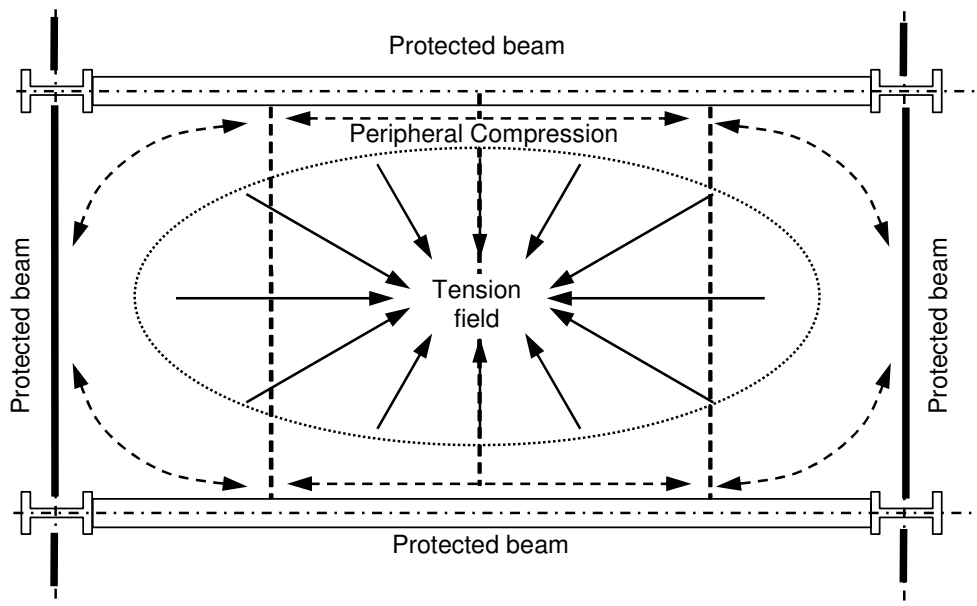


Figure 6: Typical composite floor system; tensile membrane action at high deflection.

- If temperatures continue to increase this may eventually end in a real structural collapse if the edge support fails, or may simply lead to a tensile slab fracture, either at the edges or within the middle region, which is extensive enough to constitute a compartment integrity failure.

Design based on the use of tensile membrane action

During the late 1960s significant experimental and theoretical research work by Park (1964), Sawczuk and Winnicki (1965), Kemp (1967) and Hayes (1968) was conducted on the behaviour of concrete floor slabs when subjected to large vertical displacements, when they could support loads considerably greater than those calculated using the well-established yield-line approach. The mechanism for supporting the load was shown to be tensile membrane action, which could form within the slab irrespective of whether it was restrained or unrestrained horizontally at its boundaries. A simplified design method was developed initially by Bailey (2000a, 2000b, 2001) at BRE in the United Kingdom based on this research, in the context of composite floor slabs at large displacements and subjected to elevated temperatures. By using membrane action it is possible to design composite framed structures in which a large proportion of the steel beams within a given floor-plate can be left unprotected for a defined fire resistance period. Previously, considering only flexural action, all the supporting steel beams within a given floor-plate have generally required some form of passive fire protection to achieve the required fire resistance. The method has been slightly adapted by Clifton (2001) at HERA in New Zealand.

All design using tensile membrane action embodies a basic philosophy which divides the floor-plate of a building into square or rectangular slab panels which are surrounded by protected beams (Figure 7). Even in terms of the small-deflection flexural capacity of the slabs it is easy to see that this arrangement induces them to create failure patterns which lead to the creation of larger-deflection membrane stresses.

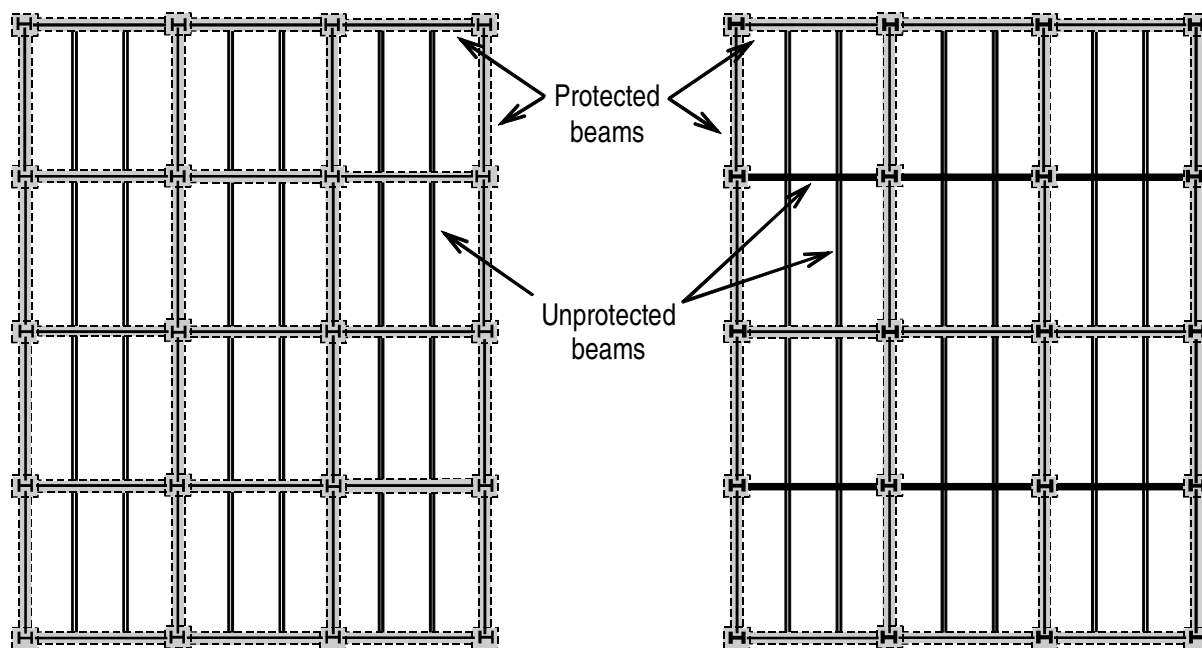


Figure 7: Passive protection strategies to take advantage of tensile membrane action.

In a floor-plate of the arrangement shown in Figure 8(a) the yield-line patterns shown in Figures 8(b) and 8(c) are possible, depending on whether adequate support is provided by the long-direction

gridline beams. In Figure 8(b) the parallel “concertina” folds do not induce any action other than bending, while in Figure 8(c) the yield-line patterns cannot develop without membrane stresses being created.

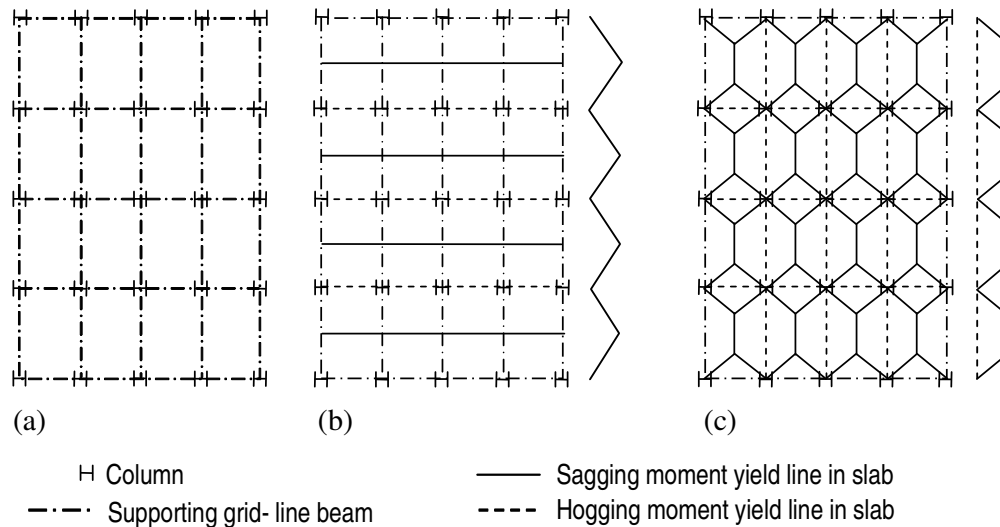


Figure 8: Possible yield-line small-deflection patterns in a composite floor plate.

Bailey’s simplified method for calculating the fire resistance of composite slabs supported on protected edges and using tensile membrane action at high deflections is presented in terms of an amplification to their small-deflection yield-line (flexural) capacity, which is greatest for square slabs and diminishes progressively as the overall length:width aspect ratio increases. In fire the method also works best when the protected (or otherwise vertically supported) areas are of a reasonably square aspect ratio, so that the double-curvature necessary to tensile membrane action is maximised. The secondary beams within each of these panels are left unprotected. During a fire the unprotected beams lose a significant proportion of their strength and stiffness which, coupled with thermal effects, results in large vertical displacements of the composite slab. The applied load is then mainly supported by membrane action in the composite slab, effectively transferring the load from the unprotected beams to the protected beams.

The proportion of beams within a given floor plate which can be left unprotected will be governed by the size and aspect ratio of the panels, and controlled by their membrane capacity. A major assumption is that the slab reinforcement over the protected beams is assumed to fracture during the fire due to the combination of high hogging moment and membrane force which occurs in this area. This is a conservative assumption since, if the reinforcement at the edges is assumed to remain intact, the induced tensile membrane forces in the slab, and thus its load-carrying capacity, will be significantly higher. In this case the slab panels are assumed to be unrestrained horizontally, so that the peripheral compressive tractions alone provide horizontal support to the tension field in the central region. The vertical support for the load carried by the slab is provided by the protected beams on the column grid, which have to be assumed to remain effectively undeflected. The critical temperature of the slab is defined as that at which a tensile integrity failure of the slab, in the form of a through-depth crack situated at the middle of its long span and running across its short span, is formed. This is identified by means of a limiting deflection based on a combination of thermal bowing and tensile straining of the long-direction mesh.

Performance-based design based on thermo-structural numerical modeling

Performance-based structural fire engineering design based on numerical modeling is now becoming increasingly feasible, particularly in the types of structure on which the research effort of the past 20 years has been concentrated. In the UK many recent projects have adopted a performance-based approach to their passive fire protection strategies. This can be especially effective where the structural fire engineering is considered as part of the overall concept of a building, integrating its requirements with those of the architecture, structure, building services and fire safety (means of escape and smoke control). Under these conditions it is possible to value-engineer the whole concept so that fire protection is placed where it is needed on the basis of an understanding of how the structure will perform in fire, and on the basis of a rational assessment of the risks. Such a process does not necessarily reduce the level of protection in any specific location, in fact it may increase the protection requirement on certain key structural elements; for example the beams which lie on the column grid may need to carry increased loads if other beams are left unprotected.

It is possible to improve on the traditional prescriptive or simple performance-based methods for specification of fire protection to all exposed steel surfaces by using three-dimensional modeling using either purpose-designed or advanced general-purpose finite element software with similar capabilities. This is often referred to as full-frame modeling, but is a more practical tool as a collection of sub-structure analyses of representative areas. A true full-frame analysis is very time-consuming, both in its set-up by engineers and in machine run-time, because of the complexity and nonlinearity of the modeling. To set up rational and conservative sub-structures taken from critical areas of the building requires a degree of conceptual awareness by the engineer, but is far more effective in dealing with a range of scenarios within a reasonable design period.

The computational process could, at some stage in the near future, involve interaction of

1. Modeling of the development of natural fire atmosphere temperatures in a range of compartments, given design distributions of fire load and agreed compartments or more open zones within the building, using computational fluid dynamics (CFD) software. This is currently the least feasible stage for practical design of whole buildings. Although CFD is now widely used to predict the movement of smoke in buildings for life-safety and escape purposes, the combustion process itself is highly complex, even if the fuel load and ventilation conditions could be predicted with confidence, and such modeling cannot be guaranteed to be reliable. At present CFD would also be extremely costly in time and computational effort, given the large number of design cases which would be required, except in very well-defined environments which generally involve localized fire scenarios in large spaces rather than flashed-over compartment fires.
2. Heat transfer analysis in order to predict the development of temperature distributions within all parts of the structure throughout the duration of the fire, and for the structure's cooling period.
3. Non-linear mechanical analysis of the structural response to the temperatures experienced.

This process should, in principle, be totally interactive rather than a one-way sequence. Structural behaviour can affect the fire development, for example by causing glass breakage, and the breaching compartment walls and other separating elements, which can also allow fire spread away from its source. However, in order to make the process feasible within a reasonable design timescale the most usual way in which it is currently conducted is:

1. Agree design cases (a range of substructures, fire loads and ventilation areas, whether to compare

with the Standard Fire curve) and acceptance criteria with the building control authority. The acceptance criteria will usually include structural stability for complete burn-out, and for the prescribed fire resistance period in the case of Standard Fires, as well as limiting deflections. Since it is important to maintain the integrity of slabs, limiting strain criteria for concrete may also be set. Of course this would need to be done in any performance-based approach.

2. For the agreed range of fire scenarios and design cases perform Eurocode 1 parametric fire analyses to establish time-temperature curves for the fire compartments.
3. Perform Eurocode 3 spreadsheet-based heat transfer calculation to produce time-temperature curves for either complete steel member cross-sections or for each plate element within these cross-sections. This is fairly reliable for unprotected steel and for steel protected with traditional passive insulating materials; in the case of intumescent protection an over-conservative approach has to be taken. For the temperature distributions in concrete slabs either a one- or two-dimensional numerical thermal analysis needs to be undertaken.
4. Perform non-linear mechanical analysis of the structural response, subject to the agreed acceptance criteria.

The designer must be aware that building control authorities are generally unfamiliar with performance-based structural fire engineering, and the use of more than one method can be advantageous in illustrating the logic of the process to them. The use of the spreadsheet-based simplified membrane action method described above can also be used as a valuable initial guide to the feasibility of proposed strategies before performing sub-structure modeling.

THE NEED TO CONSIDER ROBUSTNESS

The easy accessibility of advanced three-dimensional analytical software, including graphical interfaces which allow rapid layout changes, the capability of dealing with second-order effects, and the real feasibility of utilising torsion, has recently made it feasible for designers to specify highly optimized structural layouts for ambient-temperature design. This has enabled the realization of high-profile buildings which would not previously have been practicable, and the member optimization involved has tended to bring most member load ratios, and the failure loads for the design load cases, together, removing the excess capacity which previously existed in non-critical failure modes. The internal forces in members of such structures are inevitably highly interactive, without the structural isolation of members in simply-connected steel structures, so that local loads can cause significant effects in other parts of the structure which can be quite remote from the loads themselves. An almost inevitable outcome of such optimization is that when the load capacity of the structure is exceeded the subsequent behaviour tends rapidly to become unstable, without the prolonged ductile phase which tends to be assumed by structural engineers because of the ductility of steel itself.

A real fire very rarely needs to be considered as if the whole enclosed building volume experiences simultaneous burning, and so only some of the load-bearing structural elements are directly subjected by high surrounding temperatures. The loss of strength and stiffness of any heated part happens simultaneously with thermal expansion of its natural dimensions, which causes both local distortion and interaction with the restraint provided by the rest of the structure, particularly parts directly adjacent to the fire compartment. In most circumstances this may be considered as “load-sharing” between the directly affected zone and more remote parts, and causes the inherent fire resistance of a real building to be much higher than is indicated by methods based on isolated members. The only

way of predicting how a continuous building behaves in fire is by advanced numerical modeling which includes all the changes in material behaviour at high temperature, the distributions of temperature through parts of the structure and the ability to work accurately at very high deflections and strains. When designers have the expertise to use such methods they give the most realistic view of the way the structure will behave during a fire, and generally show that much less protection is necessary to provide the required fire resistance than the cruder methods suggest.

The consequences of local failures

Different types of local failure cause different degrees of risk to a building. Failure of a column by buckling is potentially disastrous, because the whole region of the building above it will move downwards with the top of the failing column, and even if the weight of this part of the building could be redistributed to adjacent columns then they may have to be even stronger to kill the huge amount of kinetic energy that may accumulate. The surrounding columns can be overloaded by the static and dynamic forces caused by decelerating the motion of this huge mass of building, and a progressive failure can develop rapidly. It is not sensible to leave columns unprotected because the potential consequence is so extreme.

Beams can weaken and undergo very large deflection, but provided that they remain connected at their ends, and that the attached slabs do not collapse, all the effects will usually remain local and there is no building failure. If floor slabs fail locally as a result of high deflection, the main problem is that fire can then spread upwards into the next storey; containment of a fire so that it can burn out locally is one of the main objectives of fire safety engineering. If, however, the joints between beams and columns, or between secondary and primary beams, fracture, then several undesirable effects can occur:

- The effect of the loss of integrity involved may mean that fire can spread upwards through the gap into upper storeys.
- If beams lose support then the floors supported by them may collapse onto the floors below, which may not be strong enough to carry the increased load and can start a progression of collapse of floor slabs down the building.
- A column to which the joint is connected may lose its lateral support from the beam and floor (especially if it is a perimeter column), which may increase its unsupported height (or “buckling length”), considerably reducing its buckling strength and causing it to fail.

Forces on connections during fire

Because of both the different temperatures generated in different parts of the structural system and the different thermal expansion characteristics of steel and concrete, thermally-induced stresses appear in a composite floor framing system from the time at which heating of the structure starts. In a system whose protection has been optimized to make use of tensile membrane action, by applying passive protection only to the steel downstand beams which lie on the column grid, the unprotected intermediate secondary beams attempt to expand considerably more than these protected beams from the early stages of a fire. In achieving equilibrium of a continuous floor system across the protected primary beams, particularly when there has been little loss of steel strength, this generates tensile tying forces in the connections between the protected secondary beams and the columns. These early-stage force patterns are illustrated in Figure 9.

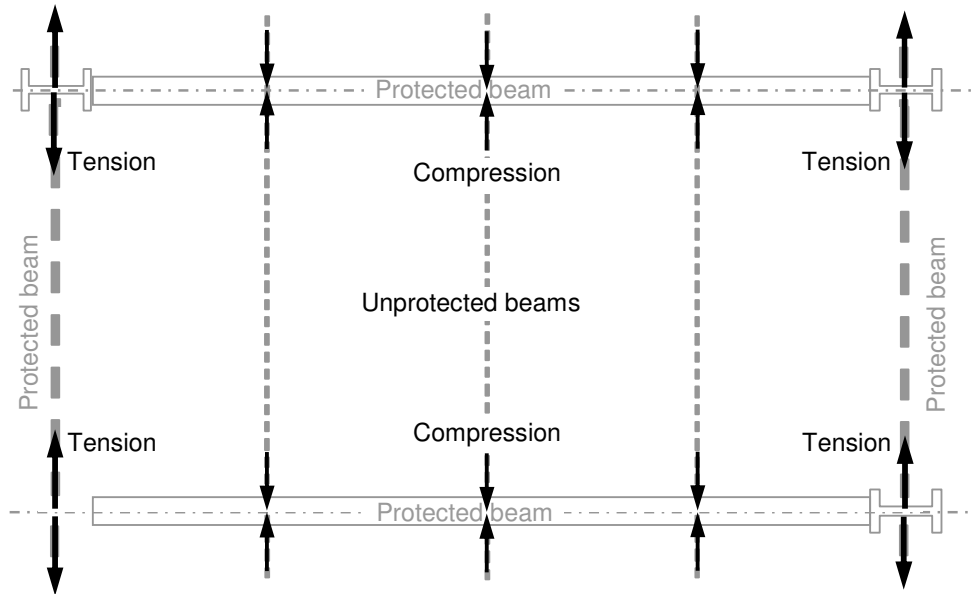


Figure 9: Early-stage connection forces, due to differential heating of steel beams.

However, as the fire temperature increases further the thermal curvature caused by the downstand beams heating much more rapidly than the attached concrete slab is to produce a net shortening which is greater than the mean expansion of the composite section. This reverses the pattern of forces across the primary beam so that the connections of the protected secondary beams to the columns now experience compressive tying forces.

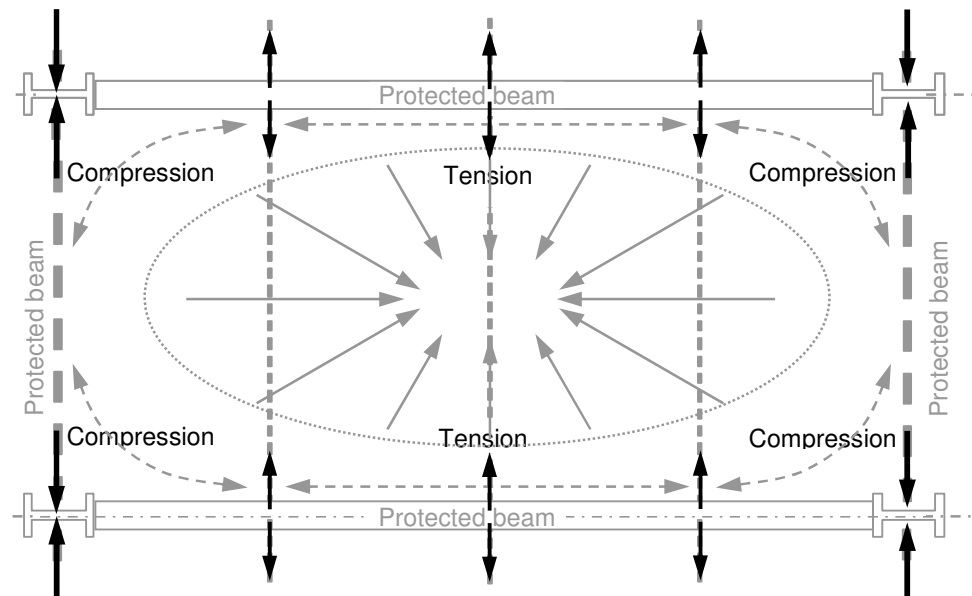


Figure 10: Connection forces due to tensile membrane action slab distortions.

When the unprotected steel temperatures rise significantly above 400°C, the balance between the importance of thermal expansion and weakening of the steel changes progressively. Whereas the previous deformation and stress patterns were predominantly thermally induced, the strength of steel is now reduced to levels at which the flexural strength of the composite members is insufficient to support their loads. If the protected primary and secondary members are still capable of providing the required vertical support, then the slabs are forced to deflect in double curvature, at which stage tensile

membrane action in the concrete slab, illustrated in Figure 6, becomes the load-carrying mechanism. While horizontal edge forces are not necessary at this stage to carry the load, the greater distortion of the slab across its middle regions than along its supports causes a greater tendency to net shortening in these regions. With continuous slabs, as shown in Figure 10, this once again causes tensile stress across the mid-span regions which are equilibrated by compressive tying forces at the connections.

Thermal insulation to the protected secondary beams can do no more than simply slow the rate at which they heat, and in any fire which remains near its peak temperature for some time these beams will eventually be above their critical temperatures as purely flexural members. These critical temperatures will usually have been reduced in any case by the extra loading shed to the protected beams from the parallel unprotected beams. The whole system of parallel secondary beams is thenceforward effectively one-way-spanning, and the floor slab hangs mainly in single curvature from its protected primary beams and the columns between them. This is catenary action (Figure 11), which is distinguished from tensile membrane action by the fact that it is not self-equilibrating, but needs to be anchored in the horizontal sense at its supporting edges.

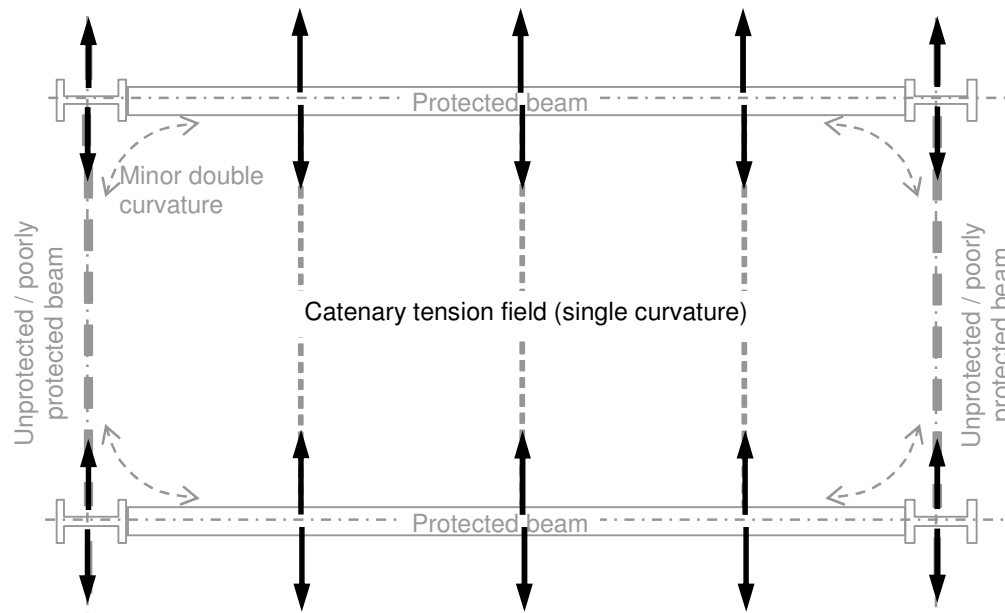


Figure 11: Connection forces due to catenary action at high deflection.

In this case integrity failure may take place due to the tension in the slab, or structural failure may occur in any of the sequence of structural components which are affected by this tension. The joints between the composite beams spanning in the direction of the catenary tension and their supporting columns may now fail due to lack of tensile tying capacity at elevated temperature. The joints are also subject to very high rotations at this stage, which may combine with the tying forces to weaken them progressively or fracture the components which are in the highest tension. At the building edge the necessary horizontal stiffness to anchor the catenary tension is mainly provided by the sway-stiffness of the perimeter columns in the fire-affected storey and the storey immediately above it. If the columns in the fire-affected level heat significantly, then this stiffness is being reduced considerably, and there is a possibility that they may buckle inwards as the heated floor sags and pulls-in. This would be a very severe form of failure, involving at least a local, and potentially an overall, structural collapse. When using catenary action as part of a structural fire engineering design approach it is necessary to check the effects of the catenary tension along its load path to a strong support, and that each structural component in this chain can resist these effects.

ROBUSTNESS OF CONNECTIONS

Structural steel connections have been extensively investigated over the past three decades to determine their moment-rotation characteristics. However, the importance of tying capacity had been realized even earlier, since the explosion at Ronan Point (Bignell *et al.* 1977) in 1968 caused progressive collapse of a large part of the building. The UK code BS5950 Part 1 (BSI 2001) requires connections to have minimum tying capacities. The SCI & BCSA (2002) design guidance checks tying capacity as an isolated action, whereas in reality a combination of tying, shear and moment usually exists. For individual bolts, resistance to tying force may be affected by co-existence with other forces. For a complete bolted connection combined actions can prevent a uniform distribution of the resultant tying force between the bolts, so they fail sequentially, reducing the tying capacity.

Connections, as the key components which tie structural members together, are important in maintaining structural integrity and preventing progressive collapse. Only limited research has been done on the performance of connections at elevated temperatures, most of which has concentrated on end-plate connections, and has mainly been confined to moment-rotation behaviour. A further complexity is that interactions between structural members during heating cause continuous changes in the forces and moments taken by the connections.

From 2005 to 2008 the universities of Sheffield and Manchester conducted a joint research programme with the aim of investigating the robustness (capacity and ductility) of steel connections at elevated temperatures. Previous tests (Ding 2007) had shown connections being subjected to tying forces from catenary action varying from 0.65 to 1.6 times their shear force at high temperatures. Hence, the investigation adopted a test setup in which the connections were subjected to a combination of tension and shear forces. Moments and rotations were generated at the connections due to the lever arm of the applied force. In total, four types of connection were studied; flush end-plates, flexible end-plates, fin plates and web cleats.

Test Setup

A detailed description of the test setup and test measurements has been given previously (Yu *et al.* 2007), so only a brief overview will be given here. The tests were performed in an electrically heated oven of 1.0m³ internal capacity, as shown in Figure 12. The specimens were heated slowly to the specified temperature, and then loaded to failure at constant temperature. A special loading system was designed to allow very large rotation of the tested connection. It includes three link bars, each connected to a central pin, with their other ends respectively connected to the jack, the specimen and a fixed hinge. When the head of the jack moves downward, it applies a tensile force to the end of the specimen through the action of the linkage. The loading jack was displacement-controlled. The applied load was measured from strain-gauges attached to the bars. Deformations of the connection were measured using a digital camera facing it through a glass window in the oven door. In all cases a UC254×89 section was used for the column, and the beam specimens were all UB305×165×40. A custom-made connector was bolted to the free end of the beam, and the load from the tie-bar was applied to this connector through a pin.

Semi-rigid connections - flush end-plates

Flush end-plate connections are widely used in the UK, and their moment-rotation characteristics have been investigated previously at ambient (Aggarwal 1994) and elevated (Al-Jabri *et al.* 2005) temperatures. Normal calculation of tying capacity assumes that the connection is subjected to pure tension, and that each bolt row contributes fully to its tying resistance. This is impossible in practice.

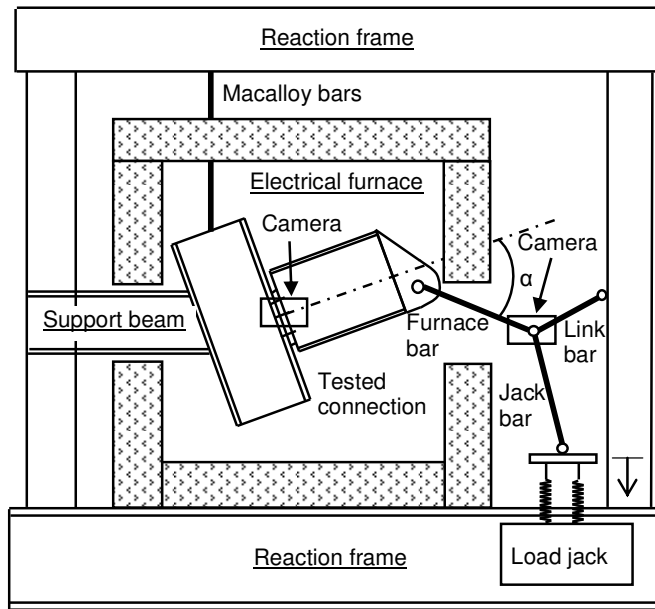


Figure 12: Test set-up.

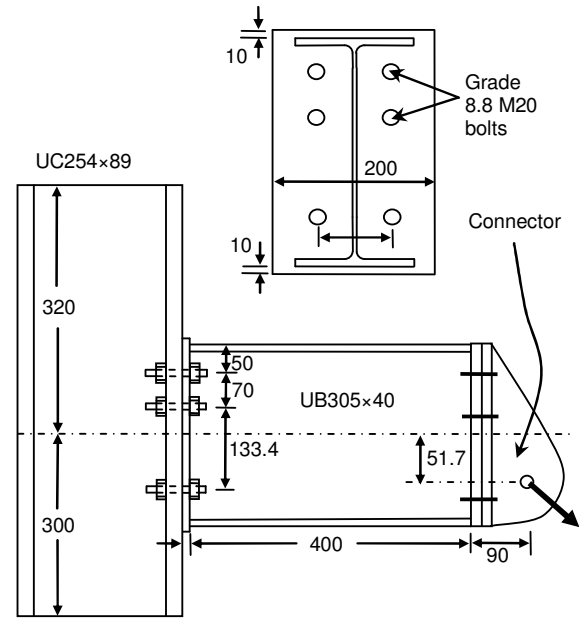


Figure 13: Typical end-plate connection.

Co-existing actions may overload individual fasteners, which indicates that all the bolt rows cannot reach their maximum resistance at the same time if their behaviour is not ductile enough, and this may cause an “unzipping” failure.

Figure 13 shows the details of a typical connection. The arrangement of the bolts is typical of current UK design, with one exception. Although it is common practice to use an end-plate 20mm thick, in order to serve the objective of developing a component-based model, it was desirable to generate various failure modes. Therefore, three end-plate thicknesses, of 8, 10 and 15mm, were tested. Most tests used three bolt rows, as shown, but for two tests the middle bolt row was removed. Connections were tested with three different combinations of shear and tying force, corresponding to different angles α in Figure 12. Three nominal initial angles α , of 55° , 45° and 35° were chosen between the axis of the steel beam and the furnace bar. Its value was monitored throughout the tests using two cameras. During each test the angle α changed progressively from its initial value, the degree of variation depending on the exact geometry of the loading system. In total 15 were successfully finished.

In these tests a relatively stable resistance was maintained up to about 7° . The resistance of the connection reduced rapidly with increase of temperature. The load angle was seen to have some effect on the overall connection resistance, but not on the failure mode. The comparison of three 550°C tests with 8mm, 10mm and 15mm thick end-plates in Figure 14 shows the main effect of end-plate thickness on the response of the connection; a thick end-plate enhances resistance but significantly reduces ductility. The effect of removing one bolt row is shown in Figure 15, comparing 2 tests with 3 rows with 2 tests with 2 rows; removing the middle bolt row clearly reduces the resistance of the connection, but is seen from the results at 550°C also to reduce the ductility.

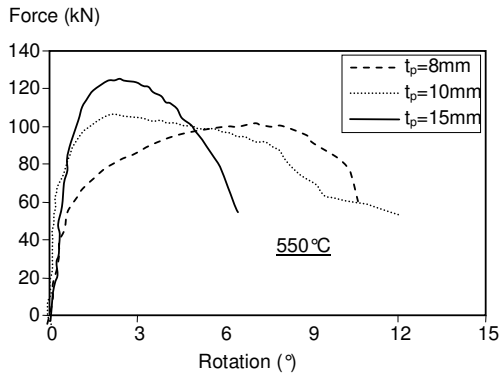


Figure 14: Effect of endplate thickness.

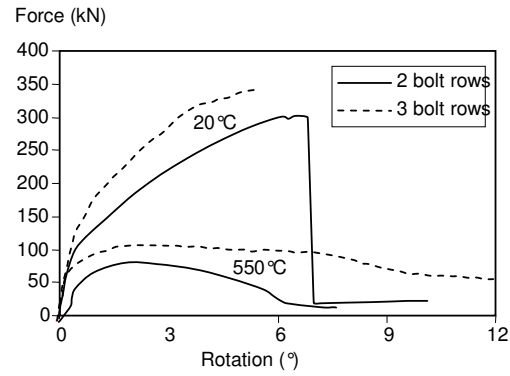


Figure 15: Effect of number of bolt rows.

For the tests with 10mm thick end-plates and three bolt rows, two failure modes were observed. At 20°C and 450°C failure was controlled by end-plate fracture. Figure 16 shows an example after a test at 450°C. This is a block-shear fracture of the end-plate in the heat-affected zone of the welds, extending from the beam web to the free edge of the beam flange, producing a sudden drop of resistance at around 7° rotation. At 550°C and 650°C, failure was controlled by the very ductile bolt extension characteristics, as shown in Figure 16. Here the end-plate is seen to have a moderate amount of bending deformation, and bolts were actually gradually pulled apart, with no obvious breaking point. When the top two bolts were completely fractured, at very large deformation, the middle two bolts had shown quite visible necking. Both rows of bolts had acquired significant bending deformations.



Figure 16: Failure of endplate connection at 450°C.

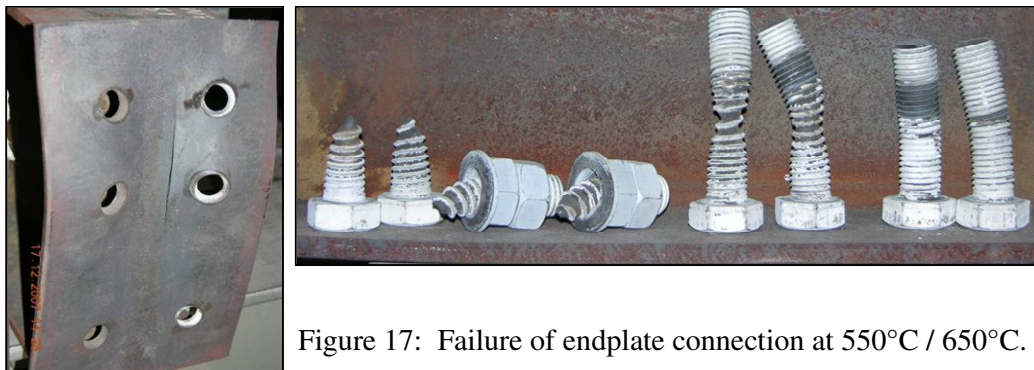


Figure 17: Failure of endplate connection at 550°C / 650°C.

For the 15mm thick end-plate the failure was, unsurprisingly, controlled by the bolts. However, compared with the joints using thinner end-plates, the bolts remained almost straight, and the end-plate was almost undeformed after the test. In the tests using two rows of bolts, bolt fracture controlled the failure in both ambient- and high-temperature tests. At 20°C, the bolts caused significant bending deformation to the end-plate before they underwent a brittle fracture. At 550°C, the end-plate remained relatively straight.

Comparison with simple connections

Similar tests were performed on commonly used simple connections, namely flexible end-plate, fin plate and web cleat connections, designed according to “Green Book” (SCI & BCSA 2002) recommendations to connect columns and beams of these sizes. The responses of these simple connections are compared with the flush end-plate connection in Figure 18. It can be seen that the major effect of the shear force is to generate a moment at the connection; the shear force itself at this level does not have a significant effect on the behaviour of the connection. At all the temperatures tested, flush end-plates clearly show higher rigidity compared to the other connection types. Their force-displacement curves are characterized by a rapid rise to peak resistance, and failure at low rotation angles.

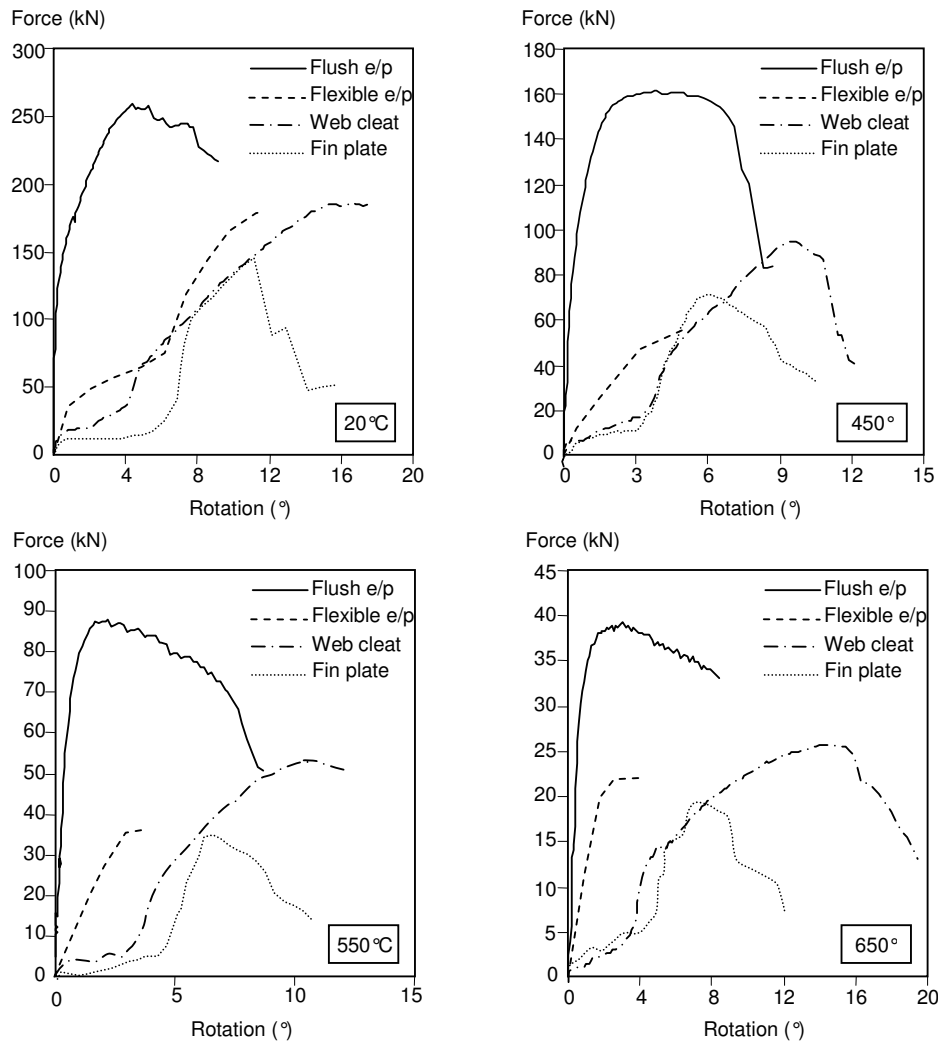


Figure 18: Comparison of the behaviour of different connection types.

All the flexible end-plate connections tested failed by fracture of the end-plate in the heat-affected zone adjacent to the welds to the beam web. This failure mode gives this connection type a very low rotational capacity at high temperatures. At ambient temperature, the performance of flexible end-plate connections is comparable to that of the other simple connections, in terms both of resistance and rotation capacity.

All the fin plate connections tested failed by shear fracture of their bolts. Bolt clearance at holes allowed the connections a rotation of up to 4° before the bearing surfaces were in contact. This gave them a rotation capacity slightly better than that of flexible end-plates. The “Green Book” states that bolt shear fracture can be avoided by limiting the thickness of the bearing plate to less than half of the bolt diameter. This proved to be inadequate at high temperatures, although at ambient temperature the bolts did cause bearing deformation of the bolt holes, and increased the rotation capacity by 2-3°. Other tests (Yu *et al.* 2009a), using Grade 10.9 bolts, successfully changed the failure mode to block-shear fracture of the beam web, and increased the rotation capacity by about 3° at ambient temperature. However, this benefit was not seen at high temperatures, since the failure was again by shear fracture of the bolts.

The web cleat connections (Yu *et al.* 2009b) failed in a more complex fashion. At ambient temperature, the bolt head punched through the angle connected to the column flange. At 450°C and 550°C, the angle fractured close to its heel at a significantly smaller deformation than at ambient temperature. At 650°C, the ductility of the angle seemed to have improved again, and failure of the connection was by shear fracture of the bolts through the beam web. At all temperatures web cleat connections showed high rotation capacity, due to the “straightening” of the angle cleats. With the increase of rotation, the load capacity increased steadily, which gave the web cleat connections a significantly higher ultimate resistance than the other simple connections.

DESIGN IMPLICATIONS

It has been seen that connections between beams and columns, as well as those between primary and secondary beams, in composite buildings can experience either tensile or compressive tying forces in fire. The sense and magnitude of the tying force in a connection can change particularly depending on the stage of the fire and the way in which the floor load is being supported at the time. The Sheffield/Manchester joint testing programme has concentrated on tensile tying forces, combined with vertical shear and rotations of the orders which might be expected when the members are at their hottest, and are carrying the floor loading in catenary action. Clearly this is only a part of the picture; if beam-to-column connections are designed to be robust enough not to fracture at this stage then the possibility of perimeter columns being pulled-in by folding floor beams must be taken into account. Equally, if the protected beams do not reach their own capacity there is still a distinct possibility that unprotected secondary beam connections to primary beams will fracture, particularly if there is continuity of the slab across the primary beams, given the very high rotations to which they are subjected. At the least this will be accompanied by fracture of the slab above the primary beams, which may easily give rise to a compartment integrity failure, permitting the fire to spread vertically. However, the avoidance of disproportionate structural collapse is paramount, and so it is clearly very important that the behaviour of the connections is understood and can be adequately represented in advanced numerical modeling used in design. The objective for designers is to obtain an adequate balance of tying strength and ductility which enables the connections to avoid fracture. In doing this the main role of tying ductility is to allow higher beam deflections in catenary, thus reducing the catenary tension forces which have to be carried through the connections to the level at which the connection strength is adequate.

The capacities in terms of moment, tying force and rotation are completely inter-dependent. Both moment capacity and tying capacity are based on the tensile behaviour of each bolt row. Increase of the connection moment will decrease its tying capacity and vice versa. The rotation of a connection is the most important influence on its strength and on its ductility in terms of movement of the beam-end relative to the column face. Semi-rigid or rigid connections, which have higher moment resistances, tend to have lower rotational capacity than some simple connections, which may limit their application

to develop catenary action. It is not necessary to consider these three parameters directly in order to establish the limit state of a connection in fire. In most cases a component-based model can provide a sufficiently accurate and practical solution to the modeling of connections in fire. Previously component-based models have been developed mainly for end-plate connections at ambient temperature, in order to generate rotational stiffnesses and moment capacities for semi-rigid frame design. The Sheffield group has now conducted several successive research projects on steel connection behaviour in fire, culminating in the test program and numerical modeling of this research project. The behaviour of most components of the four connection types tested has been represented in simplified high-temperature non-linear spring models of the type illustrated in Figure 19.

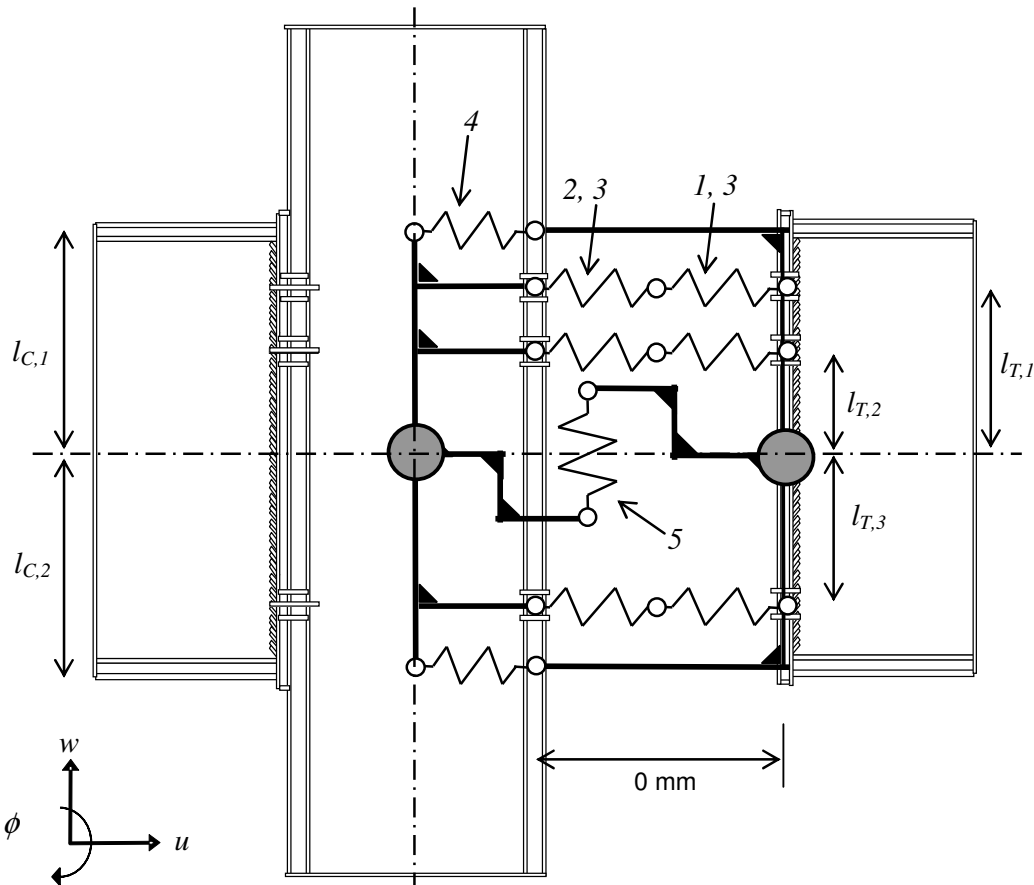


Figure 19: Schematic diagram of a component-based beam-column connection model.

Because of the need to emphasize the issue of robustness in fire, it is necessary for these models to have two innovative characteristics:

1. The pre-peak part of the load-displacement curve for a component can be as important as the peak resistance, since uniform distribution of displacement to all the bolt rows is unlikely.
2. Formation of a yielding mechanism is not necessarily synonymous with fracture. The behaviour of each component up to large deflection or fracture is necessary.

Following these basic principles, component-based models have been developed for use in modeling the four types of connection tested, and these have been shown to predict the connection behaviour with satisfactory accuracy.

A few general conclusions may be drawn about the behaviour of these connections:

1. The rotational capacity of a connection is mainly determined by the deformational ductility of its components, together with the lever arm between the top bolt row and the compressive fulcrum, which is usually near the bottom flange of the beam. The ductility of these components can be affected by temperatures.
2. The strength of bolts is reduced more rapidly at high temperatures than that of hot-rolled steel. For some connections, whose ductility is maintained by avoiding bolt failure, this must be considered for robustness in fire, and the use of stronger bolts than are needed for ambient-temperature design is recommended.
3. For fin plate connections, details which can increase ductility include using stronger bolts and higher bolt hole clearances. Tests have shown that, if the failure mode can be changed from bolt shear fracture to plate block shear, the rotational capacity can be increased considerably.
3. For flush end-plate connections, ductility is mainly due to bending deformation of the end-plate. Hence, thin end-plates should be used where ductility is required.

For all the connections, ductility is enhanced if the bolt rows are placed as close to the lower flange of the beam as possible. However, this also causes a significant sacrifice of moment capacity. More detailed studies are needed on failure of bolts at high temperature, both in shear and in tension. The resistance of bolts was observed to reduce gradually with progressive deformation, after reaching a fairly early peak. Simple models have been proposed by Yu *et al.* (2009c, 2009d) to account for this behaviour. In general, there is a lack of reliable criteria for the final fracture of steels at elevated temperatures, and this information is crucial if the robustness of steel framed structures in fire is to be investigated analytically using modeling

Acknowledgment: The author gratefully acknowledges the support of the Engineering and Physical Sciences Research Council of the United Kingdom under Grant EP/C510984/1. Provision of the steel sections by Corus Ltd is also gratefully acknowledged.

REFERENCES

- Aggarwal A.K. (1994), "Comparative Tests on End-plate Beam-to-Column Connections", *J. Construct. Steel Research*, **30**, pp 151-175.
- Al-Jabri K.S., Burgess I.W., Lennon T., Plank R.J. (2005), "Moment-rotation-temperature curves for semi-rigid joints", *J. Construct. Steel Research*, **61**, pp 281-303.
- ASFP (2007), *Fire protection for structural steel in buildings - 4th Edition*. Association for Specialist Fire Protection, Aldershot, UK.
- Bailey, C. G. (2001) "Membrane action of unrestrained lightly reinforced concrete slabs at large displacements" *Engineering Structures*, **23**, 470-483.
- Bailey, C. G. and Moore, D. B. (2000a) "The structural behaviour of steel frames with composite floor slabs subject to fire: Part 1: Theory" *The Structural Engineer* **78** (11), 19-27.
- Bailey, C. G. and Moore, D. B. (2000b) "The structural behaviour of steel frames with composite floor slabs subject to fire: Part 2: Design" *The Structural Engineer* **78** (11), 28-33.
- Bignell, V., Peters, J., and Pym, C. (1977), *Catastrophic failures*, Open University Press, Milton Keynes.

- British Steel (1999), *The behaviour of multi-storey steel framed buildings in fire: A European joint research programme*, Swinden Technology Centre, British Steel plc, Rotherham, UK.
- BSI (2001), *BS5950: Structural Use of Steelwork in Building - Part 1: Code of practice for design- Rolled and welded sections*, British Standards Institution, London.
- BSI (2000), *BS5950:Part 8: Code of Practice for Fire Resistant Design*, British Standards Institution, Basingstoke, UK.
- CEN (2002), *Eurocode 1: Actions on Structures – Part 1.2: General actions – actions on structures exposed to fires*, European Committee for Standardization.
- CEN (2005a), *Eurocode 3: Design of Steel Structures – Part 1.2: General Rules – Structural Fire Design*, European Committee for Standardization.
- CEN (2005b), *Eurocode 4: Design of Composite Steel and Concrete Structures – Part 1.2: General Rules – Structural Fire Design*, European Committee for Standardization.
- Clifton, C. (2001), “Design of multi-storey steel framed buildings with unprotected secondary beams or joists for dependable inelastic response in severe fires.” *Steel Design and Construction Bulletin* **60**, New Zealand Heavy Engineering Research Association (HERA), 1-58.
- Ding, J. (2007), *Behaviour of Restrained Concrete Filled Tubular Columns and Their Joints in Fire*, Ph.D Thesis, University of Manchester, UK.
- Hayes, B. (1968). “Allowing for membrane action in the plastic analysis of rectangular reinforced concrete slabs”, *Magazine of Concrete Research* **20** (65), 205-212.
- ISO (1985), *ISO 834: Fire Resistance Tests - Elements of Building Construction*. International Organisation for Standardisation.
- Kemp K. O. (1967), “Yield of a square reinforced concrete slab on simple supports, allowing for membrane forces”, *The Structural Engineer* **7** (45), 235-240.
- Newman, G.,M., Robinson, J.,T., Bailey, C.,G. (2004), *Fire Safety Design: A New Approach to Multi-Storey Steel-Framed Buildings*, The Steel Construction Institute, Ascot.
- NIST (2005), *Final Report on the Collapse of the World Trade Center Towers*, National Institute of Standards and Technology, USA.
- NIST (2008), *Final Report on the Collapse of World Trade Center Building 7*, U.S. Department of Commerce and National Institute of Standards and Technology, USA.
- Park, R. (1964), “Tensile membrane behaviour of uniformly loaded rectangular reinforced concrete slabs with fully restrained edges”, *Magazine of Concrete Research* **16** (46), 39-44.
- Sawczuk, A. and Winnicki, L. (1965) “Plastic behaviour of simply supported reinforced concrete plates at moderately large deflections”, *International Journal of Solids and Structures* **1**, 97-111.
- SCI & BCSA (2002), *Joints in steel construction, Simple connections*, The Steel Construction Institute and British Constructional Steelwork Association, UK.
- Yu, H.X., Burgess, I.W., Davison, J.B. and Plank, R.J. (2007), “Experimental Investigation of the Behaviour of Fin Plate Connections in Fire”, Proc. ICSCS 2007, Manchester, pp 541-548.
- Yu H.X., Burgess I.W., Davison J.B., and Plank R.J., (2009a), Experimental Investigation of the Behaviour of Fin Plate Connections in Fire, *J. Construct Steel Research*, **65**, pp 723–736.
- Yu H.X., Burgess I.W., Davison J.B., and Plank R.J., (2009b),”Tying Capacity of Web Cleat Connections in Fire Part 1: Test and Finite Element Simulation”, *Engineering Structures*, **31** (3), pp 651-663.
- Yu, H.X., Burgess, I.W. Davison, J.B., and Plank, R.J. (2009c), “Tying Capacity of Web Cleat Connections in Fire. Part 2: Development of Component-Based Model”, *Engineering Structures*, **31** (3), pp 697-708.
- Yu, H.X., Burgess, I.W., Davison, J.B. and Plank, R.J. (2009d), “Development of a Yield-Line Model for Endplate Connections in Fire”, *J. Construct. Steel Research*, **65** (6), pp1279-1289.