



The robustness of steel connections in fire

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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. Traditionally, connections have been studied in terms of their moment-rotation behaviour alone. Recently however, some work has been done to investigate 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 has been studied in furnace tests at different temperatures. This 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-based approach.

KEYWORDS. Connections; Steel; Robustness; Fire; Tying forces; Rotations; Performance-based design.

INTRODUCTION

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 [1] in 1968 caused progressive collapse of a large part of the building. The UK structural steelwork design code BS5950 [2] now requires connections to have minimum tying capacities. The UK SCI/BCSA design guidance [3] checks the tying capacity as an isolated action, whereas in reality a combination of tying force, shear force 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, causing them to fail sequentially, significantly reducing the tying capacity.

In design for fire resistance, the increasing adoption of performance-based design principles means that structures are now treated integrally in structural fire safety design. Connections, as the key components which tie structural members together, are important in maintaining structural integrity and preventing progressive collapse. Evidence from the collapse of the WTC buildings [4, 5] and full-scale fire tests at Cardington [6] have shown that connections are vulnerable to fracture in fire. 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.



The Universities of Sheffield and Manchester have conducted a joint research programme with the aim of investigating the capacity and ductility of steel connections at elevated temperatures. A recent trend in the design of composite floor systems has been to fire-protect beams on the main column grid, while leaving other beams unprotected. The protected beams eventually deflect considerably under the combined effect of high steel temperatures and enhanced loading, shed from the unprotected members, and will impose high tying forces on their connections. In non-composite steel construction, the beams deflect at high temperatures and experience catenary tension, which is transferred to the supporting structure through the connections. Previous tests by Ding [7] showed that connections can be subjected to tying forces varying from 0.65 to 1.6 times their shear force at high temperatures. Hence, the current 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 by Yu et al. [8], 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 Fig. 1. 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. The deformations of the connection were measured using a digital camera facing the connection 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.

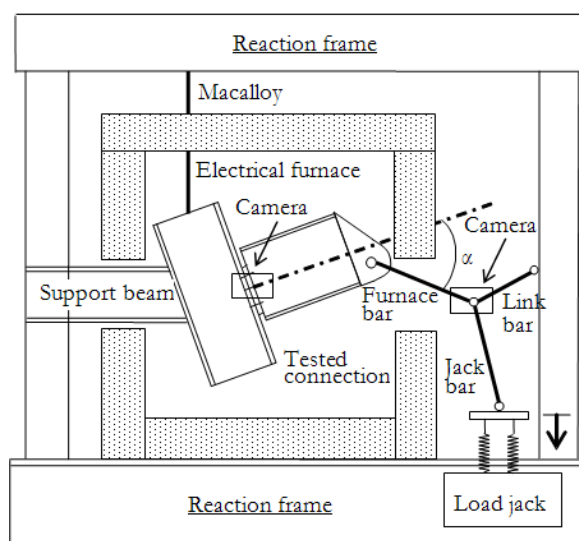


Figure 1: Test setup.

SEMI-RIGID CONNECTIONS: FLUSH END-PLATES

Flush end-plate connections are widely used in the UK, and their moment-rotation characteristics have been investigated [9, 10] previously at ambient and elevated temperatures. Normal calculation of the tying capacity assumes that the connection is subjected to pure tension, and that each bolt row can contribute fully to the



resistance of the connection. This is obviously impossible in practice. 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.

Fig. 2 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 in Fig. 2, but for two tests the middle bolt row was removed.

Connections were tested at three different combinations of shear and tying force, corresponding to different angles α in Fig. 1. 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, 17 tests were planned, but only 15 were successfully finished. In two cases the loading equipment failed and hence the maximum resistance was not achieved.

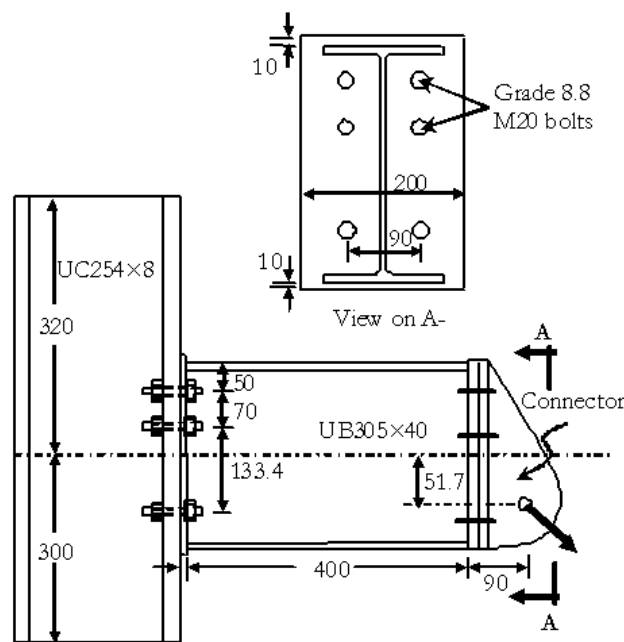


Figure 2: Typical end-plate joint.

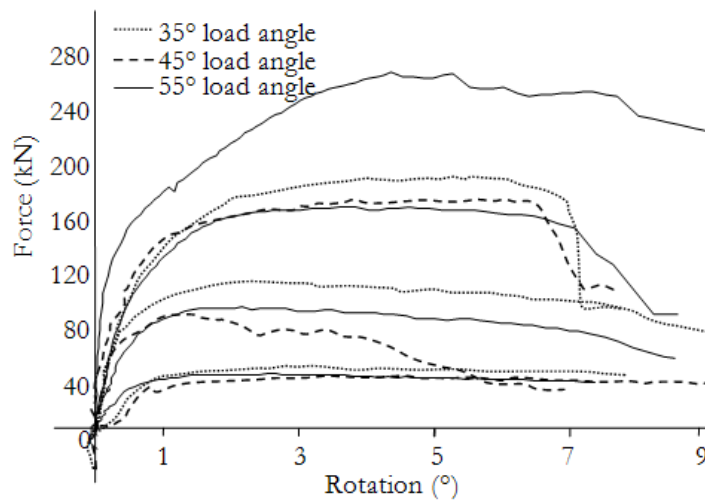


Figure 3: Force-rotation for 10mm end-plate connections.

The force-rotation relationships for the tests using 10mm end-plates are shown in Fig. 3. At 550°C, the test at 45° was the first in this series. During this test the bolts failed by thread-stripping from the nuts. For this reason the force decreases progressively after 1.5° rotation. After this test two nuts were used on each bolt to prevent thread-stripping. The other tests were able to maintain a relatively stable resistance up to about 7°. The resistance of the connection reduced rapidly with increase of temperature. The load angle has some effect on the overall connection resistance, but not on the failure mode.

Comparison of three 550°C tests with 8mm, 10mm and 15mm in Fig. 4 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 Fig. 5, 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.

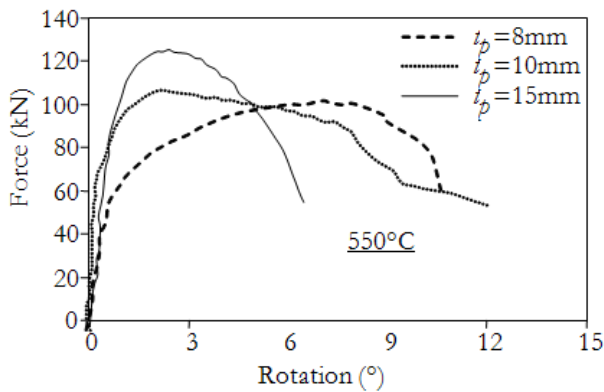


Figure 4: Effect of end-plate thickness.

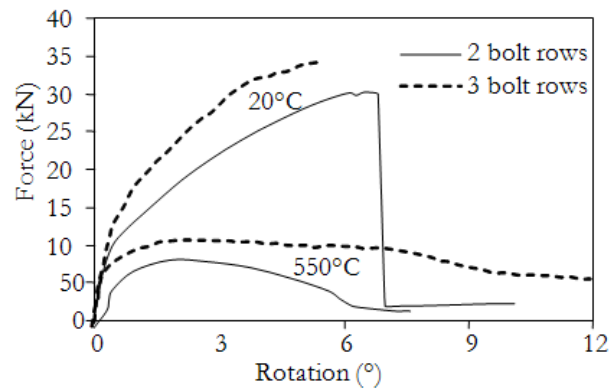


Figure 5: Effect of number of bolt rows.

For the tests with $t_p=10\text{mm}$ and three bolt rows, two failure modes were observed. At 20°C and 450°C failure was controlled by end-plate fracture; Fig. 6 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 Fig. 7. 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 6: Failure of end-plate connection at 450°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.



Figure 7: Failure of end-plate connection at 550°C / 650°C.

COMPARISON WITH SIMPLE CONNECTIONS

Similar tests have been performed on commonly used simple connections, namely flexible end-plate, fin plate and web cleat connections, designed according to “Green Book” [3] recommendations to connect columns and beams of these sizes. The responses of these simple connections are compared with the flush end-plate connection in Fig. 8. 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. Hence, the initial stiffnesses of the curves in Fig. 8 are also their rotational stiffnesses. At all the temperatures tested, flush end-plates clearly show 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.

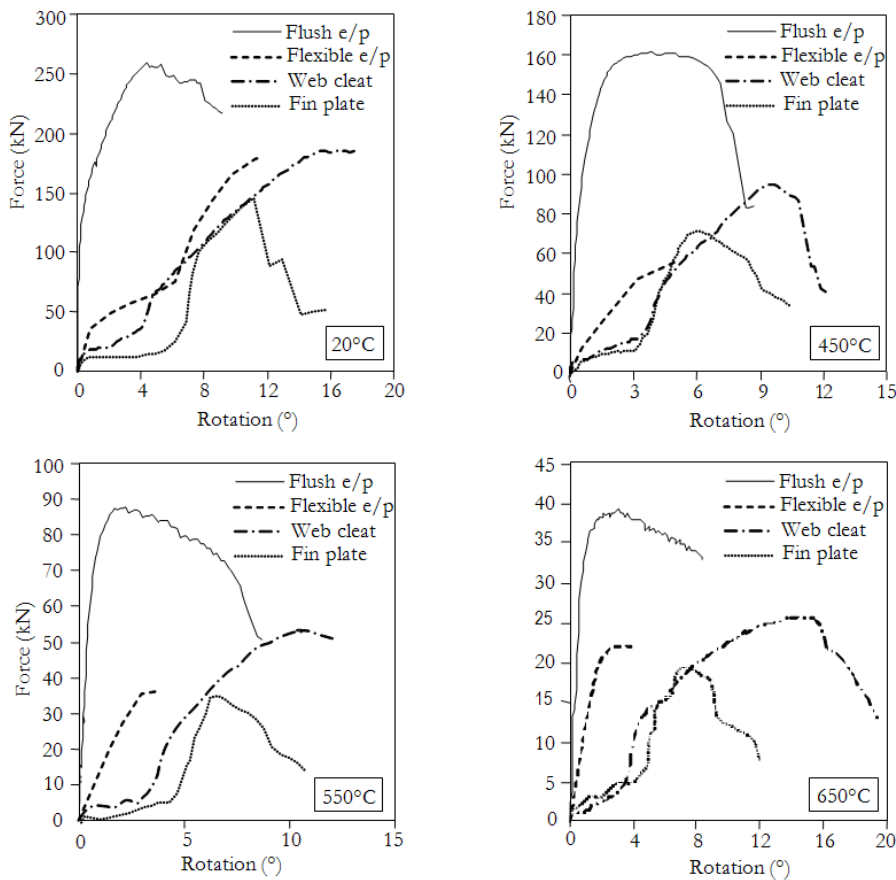


Figure 8: Comparison of the behaviour of different connection types.

All the flexible end-plate connections tested [11] 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 very low rotational capacity at high temperatures. At ambient temperature, the performance of flexible end-plate connections is comparable to that of other simple connections, in terms both of resistance and rotation capacity.

All the fin plate connections tested [12] failed by shear fracture of their bolts. Bolt clearance at holes allowed the connection 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, 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 is not seen at high temperatures, since the failure is again by shear fracture of the bolts.

The web cleat connections [13] 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.

ASSEMBLY OF COMPONENTS

Fig. 9 shows the layout of the component assembly within the component-based element. The assembly is developed to deal with a maximum of 5 bolt rows in a connection. The assembled element has 2 external nodes; internally it consists of 5 tension bolt rows and 2 compression spring rows. Node 1 is located at the intersection between the beam and column reference axes. Node 2 is the end-node of the beam. The shear components are not included in this assembly, and are assumed to be rigid in the vertical shear direction.

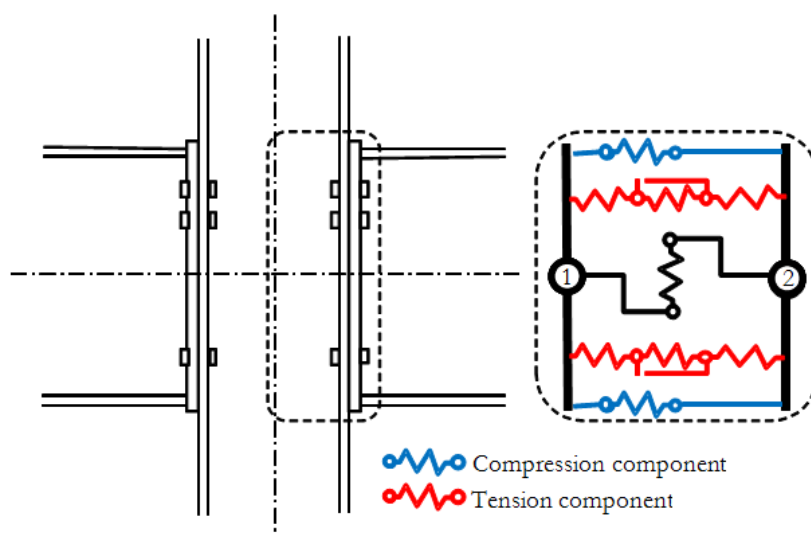


Figure 9: Component assembly.

Tension component

Each tension bolt row includes three tension components, which work in series. The middle component in each series is designed to represent the bolt in tension. The other two tension components are: the column flange in tension and end-plate in tension for flush end-plate connections; or reverse channel in tension and end-plate in tension for reverse channel connections. Each tension component's Force/Displacement behaviour is represented by a multi-linear curve without a 'descending part' (negative stiffness). The typical multi-linear curve (Fig. 10) consists of 5 points, which can be expanded



if more points are required, to plot relevant aspects of the component behaviour. Point 5 is the ‘ultimate strength’, at which point the component has its maximum resistance and deformation. Beyond this point the component is deemed to be fractured. If point 5 is reached, the whole spring row in which this component is located will be ‘switched off’ and will not contribute to the joint’s resistance or stiffness in the subsequent analysis.

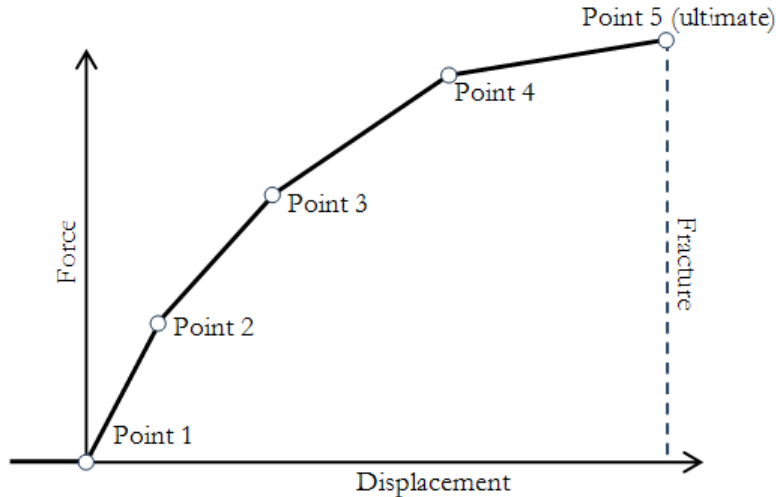


Figure 10: Typical tension component force/displacement behaviour.

Typical tension bolt row

The three components in each tension bolt row are combined into one effective spring at each temperature step (Fig. 11). The force-displacement curves of the tension bolt rows are used to derive the connection’s local force and stiffness. After the global analysis reaches a converged stable equilibrium, the forces in the tension bolt rows are established, and the displacements of each tension component are calculated. The related information, such as each component’s permanent deformation, is then updated. The maximum resistance of the effective spring is defined by the weakest component in this series. Any force above the weakest component’s ultimate resistance is ignored. At each force level, the effective spring’s displacement is the total of the components’ displacements under this force level.

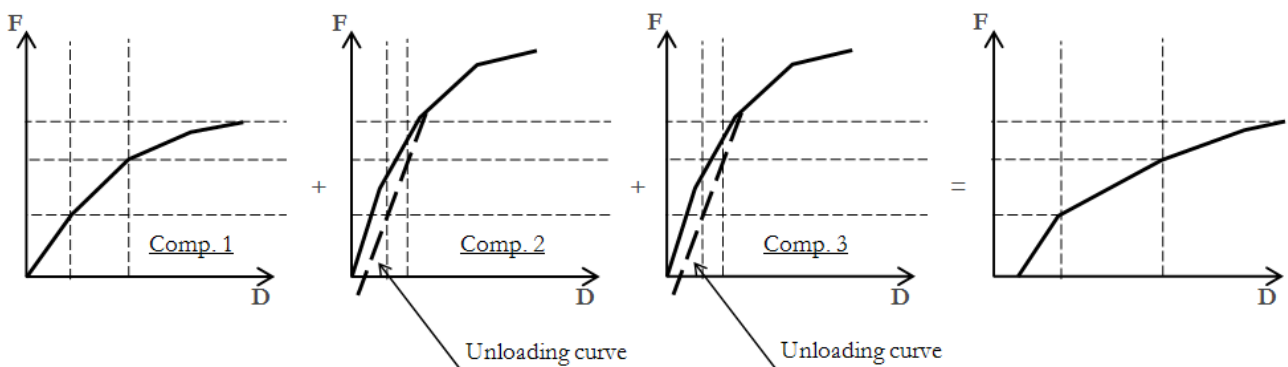


Figure 11: Assembly of the individual tension components to tension bolt row.

The typical tension bolt row Force/Displacement (F/D) curve (Fig. 12) consists of four parts: tension, bolt plastic deformation, compression, and residual compression. Each part will be used at different stages of a fire.

In an end-plate connection, the three tension components in each tension bolt row include the column flange under tension, the bolt in tension and the end-plate in tension.

- ✓ The ‘tension’ part shows all three components under tension.
- ✓ The ‘compression’ part represents the state when the plastically deformed end-plate and column flange are pushed back until their centres come into contact.



- ✓ The ‘bolt plastic deformation’ part represents the stage between the ‘tension’ and ‘compression’ parts, at which the applied tension reduces to zero, and the deformed end-plate and column flange have not yet come into contact. As the bolt-in-tension component does not work under compression, the tension spring row’s stiffness is zero during the ‘bolt plastic deformation’ part.

The ‘residual compression’ part is based on the assumption that the tension bolt row can transfer a certain amount of compression force. However, as the parts of the connection represented by the compression spring group are much stiffer than this, most of the compression force will be transferred by the compression spring group.

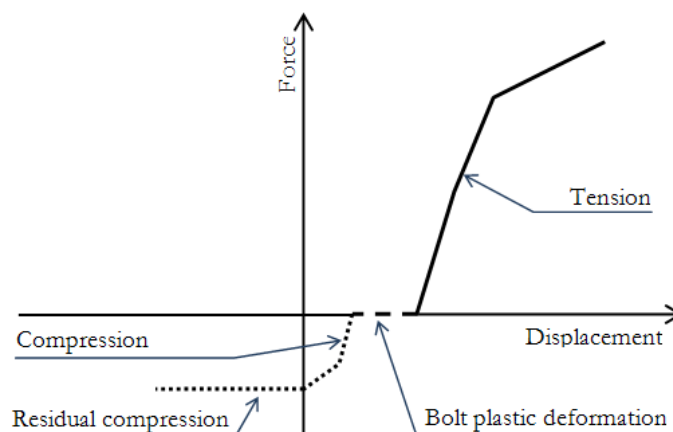


Figure 12: Effective force/displacement curve of a typical tension bolt row.

Compression spring row

A compression component is represented by 3 points, which can be expanded if necessary (Fig. 13). A compression component will be ‘switched off’ under tension force, when its stiffness contribution is zero and it transfers zero tension force. Point 3 is the ultimate strength point, beyond which it is assumed that no change of resistance occurs.

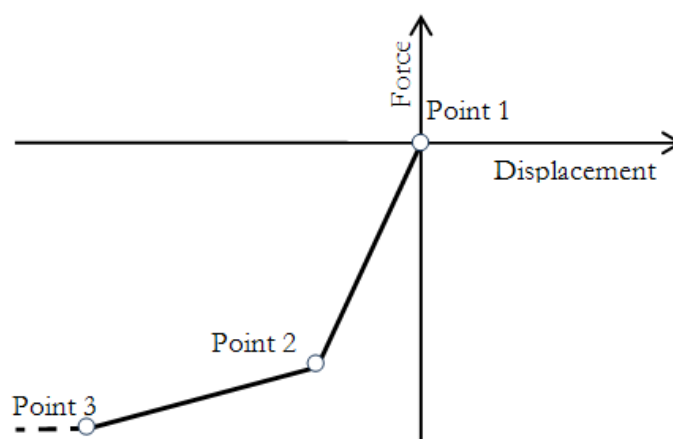


Figure 13: Typical compression component force/displacement behaviour.

Effective Force/Displacement curve of a component at constant temperature

When a component carries a force it may become inelastic and will then have acquired irreversible deformation (‘residual deformation’) when its force is reduced to zero. In this development, the classic Masing rule [14] is employed for this ‘memory effect’. The unloading curve is the original loading curve doubled and rotated by 180°. If the initial loading curve is represented [15] by:

$$D = f(F) \tag{1}$$

then the unloading curve can be described as



$$\frac{(D_A - D)}{2} = f\left(\frac{F_A - F}{2}\right) \quad (2)$$

Where D_A and F_A (as shown in Fig. 14) are respectively the displacement and force at which unloading occurred. In Fig. 1 the node (F_A, D_A) is called the Intersection Point, and the intersection of the unloading curve with the zero-force axis is called Reference Point 1, which represents the permanent deformation caused. If the applied force, or the component's displacement, is beyond its intersection point, its displacement and applied force lie on the loading curve, and its permanent displacement will increase accordingly. On the other hand, if they lie below the intersection point, then they are on the unloading curve, and the permanent deformation will not change.

Fig. 15 shows how the loading and unloading curves form the 'effective' F/D curve represent the component's behaviour.

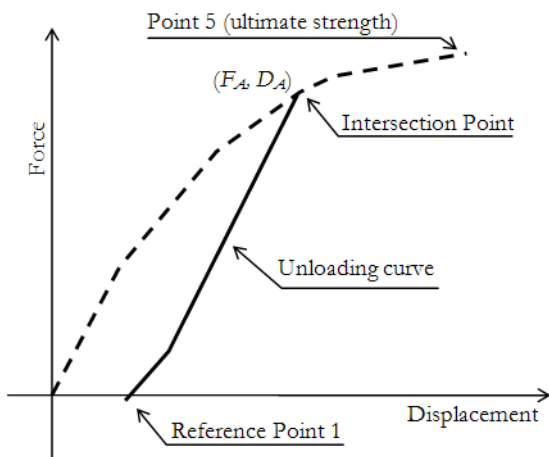


Figure 14: Typical tension component F/D curve.

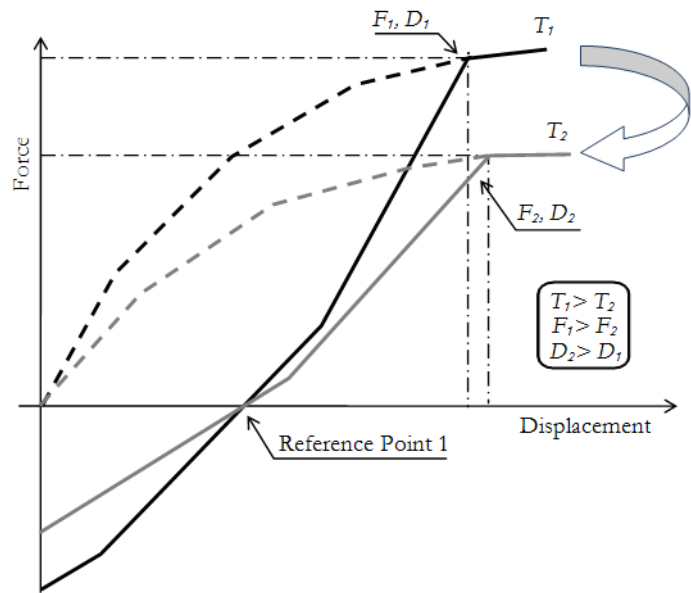


Figure 15: Unloading at changing temperatures.

Unloading with changing temperatures

When a component is heated in a fire, its F/D curve is temperature-dependent, and this temperature changes continuously during the fire. The 'Reference Point' concept is introduced to locate the unloading curve. Block [15] describes the principle of the 'Reference Point' concept as 'plastic strain which is not affected by a temperature variation'. Therefore, the component's permanent deformation is assumed not to change when only the temperature changes. When moving to the next temperature step, the component's current permanent deformation is that saved from the last step, and the permanent deformation will be updated at the end of each calculation step.

Fig. 15 shows how this concept is implemented in software. Reference Point 1 is updated at the end of the step at temperature T_1 , on the basis of the applied force and the component's Force/Displacement curve. When moving to the next step (temperature T_2), the unloading curve is plotted on the basis of the component's new F/D curve. Therefore the new unloading curve will be located by starting from a point on the new loading curve and passing through Reference Point. Finally, the effective F/D curve is formed for this temperature.

Analytical Implications

Because of the nature of conventional quasi-static analysis, an analysis of a structure in fire which includes component-based connection elements can only trace the behaviour of a connection up to the point where its first component fails. In reality a connection may either be able to regain its capacity after the initial fracture of a component, or the first failure may trigger a cascade of failures of other components, leading to complete detachment of the connected member. This possibility should be considered in performance-based design when a structure is being tested for robustness. If connections are to avoid the possibility of becoming detached from members, this numerical modelling must be capable of predicting the sequence of failures of components, rather than simply the first loss of stability. A numerical procedure

in which the whole behaviour, from first instability to total collapse, can be modelled effectively, has been developed in *Vulcan*.

The *Vulcan* model combines alternate static and dynamic analyses, in order to use both to best advantage. Static analysis is used to follow the behaviour of the structure at changing temperature until instability happens; beyond this point an explicit dynamic procedure is activated to track the motion of the system until stability is regained. When combined with the parallel development of general component-based connection elements, this procedure can effectively track the behaviour of connections, from the initial fracture of a component, via the failure of successive bolt-rows, to final detachment from the column. In fact the analysis of a simple frame model depicted in Fig. 16 carries on beyond connection fracture row-by-row, including complete detachment of the heated beam, until final structural collapse of the frame occurs due to column buckling at a higher temperature.

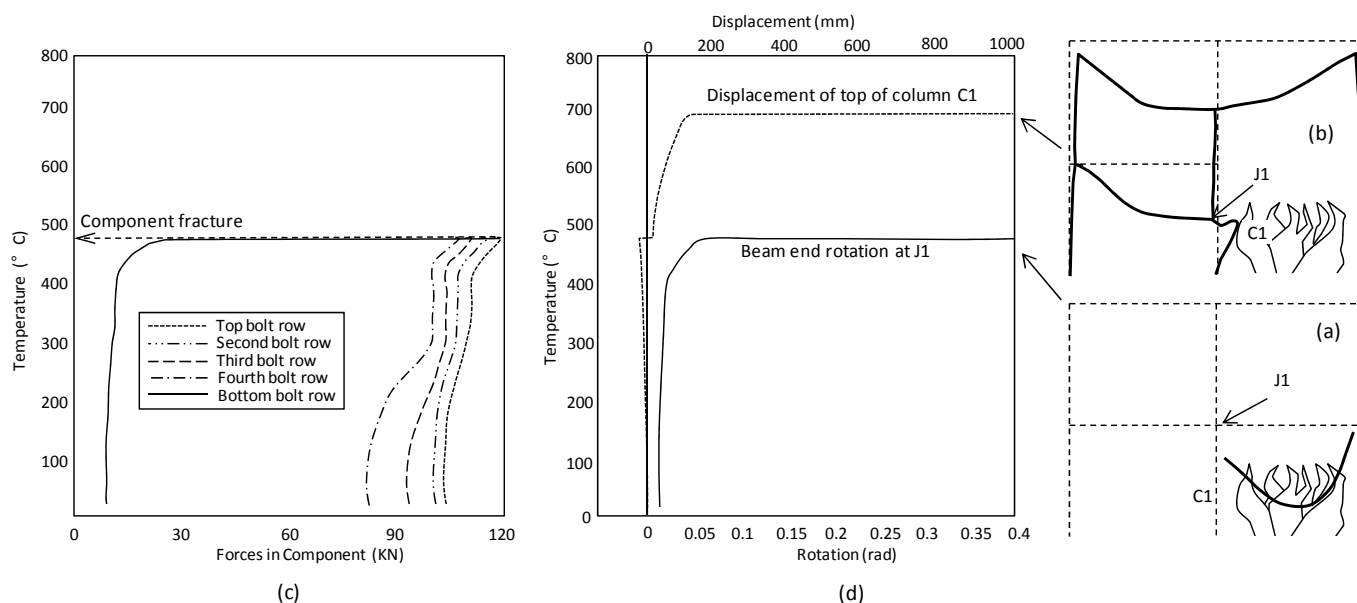


Figure 16: Progressive failure of connection with elevated temperature.

CONCLUSIONS

Connections within a structural sub-frame, if heated together with the beams that they support, will initially be subjected to compressive force due to the restrained thermal expansion of the beams. The magnitude of this force depends on the span of the beam and the axial restraint stiffness provided by the connections themselves and adjacent structure. Some connections, such as fin plates, can fail due to this force. Although this has been suggested as the cause of failure of WTC 7 [5], it has never been observed in the UK, probably because the multi-storey composite-framed structures typical of UK practice provide less axial restraint to beams. End-plate connections cannot fail under compressive forces; the compression will continue to increase until the beam reaches its limit capacity under the combined effect of the bending moment and compression. It then experiences a rapid increase of deflection, which attenuates the compression force to a limiting value as the thermal expansion is accommodated by the deflection. With further temperature increase the progressive reduction of steel strength decreases the compressive force, to the extent that the axial component eventually becomes tensile. This tension increasingly takes over from the bending resistance of the beam in carrying the loads by “catenary action”. At this stage the upper bound to the tensile force is given by the lower of the reduced strengths, at the appropriate temperatures, of the beam or its connections. This has been demonstrated in small-scale structural frame tests by Ding [7]. Ductile design of connections is important because the catenary force is reduced with increase of the deflection which is allowed by movement at the ends of the beams; some of this movement can be allowed by the connections themselves.

The capacities of a connection in terms of moment, tying force and rotation are completely inter-dependent. Both moment and tying capacity are based on the tensile behaviour of each bolt row. The rotation of a connection, in terms of



movement of the beam-end relative to the column face, is the most important influence on its strength and ductility. Semi-rigid or rigid connections, which have higher moment resistances, generally have lower rotational capacity than simple connections, which may limit their ability 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 programme and numerical modelling reported here. The behaviour of most components of the four connection types tested has been represented in simplified high-temperature non-linear spring models. Because of the need to emphasize the issue of robustness in fire, it is advantageous for these models to have two innovative characteristics:

1. A model of the pre-peak part of the load-displacement curve for a component is needed, 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 actual fracture is necessary.

Following these basic principles, components have been characterized for use in modeling the four types of connection studied in this project, and these have been shown to predict the connection behaviour with satisfactory accuracy. A general-purpose component-based connection element has been assembled which can accommodate the appropriate components within its bolt-rows, and this development has been made in parallel with a static/dynamic solution process for the *Vulcan* software. It has been shown that the behaviour of a structural frame can be modelled throughout the duration of a fire with this combination, so that transient failures of parts of connections do not cause an end to the analysis, and re-stabilization is shown if it occurs. This kind of analysis will be necessary in future for true performance-based analytical design of framed buildings against fire, so that potential disproportionate collapse can be predicted and the design of the structure, including that of its connections, can be adjusted to reduce this possibility.

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REFERENCES

- [1] V. Bignell, J. Peters, C. Pym, Catastrophic failures, Open University Press, Milton Keynes, (1977).
- [2] BS5950, Structural Use of Steelwork in Building- Part 1: Code of practice for design-Rolled and welded sections, British Standards Institution, London, (2001).
- [3] SCI & BCSA, Joints in steel connection, Simple connections, The Steel Construction Institute and British Constructional Steelwork Association, UK, (2002).
- [4] NIST, Final Report on the Collapse of the World Trade Center Towers, National Institute of Standards and Technology, USA, (2005).
- [5] NIST, Final Report on the Collapse of World Trade Center Building 7, U.S. Department of Commerce and National Institute of Standards and Technology, USA, (2008).
- [6] G., M. Newman, J. T. Robinson, C.G. Bailey, Fire Safety Design: A New Approach to Multi-Storey Steel-Framed Buildings, The Steel Construction Institute, Ascot, (2004).
- [7] J. Ding, Behaviour of Restrained Concrete Filled Tubular Columns and Their Joints in Fire, Ph.D Thesis, University of Manchester, UK, (2007).
- [8] H. X. Yu, I. W. Burgess, J. B. Davison, R. J. Plank, Journal of Structural Engineering, ASCE, 137 (1) (2011) 80.
- [9] A.K. Aggarwal, J. Construct. Steel Research, 30, (1994) 151.
- [10] K. S. Al-Jabri, I. W. Burgess, T. Lennon, R. J. Plank, J. Construct. Steel Research, 61 (2005) 281.
- [11] Y. Hu, J. B. Davison, I.W. Burgess, R. J. Plank, International Journal of Steel Structures, 9 (2009) 29.
- [12] H. X. Yu, I. W. Burgess, J. B. Davison, R. J. Plank, J. Construct. Steel Research, 65, (2009) 723.
- [13] H. X. Yu, I. W. Burgess, J. B. Davison, R. J. Plank, Engineering Structures, 31 (3) (2009) 651.



- [14] G. Masing, *Wiss Veroffentl aus dem Siemens-Konzern*, 31 (1923) 231 [in German].
- [15] F. M. Block, *Development of a Component-Based Finite Element for Steel Beam-to-Column Connections at Elevated Temperatures*, PhD thesis, University of Sheffield, (2006).