Connection Modelling in Fire

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ABSTRACT: This paper describes the origins and development of component-based principles for modelling of the behaviour of beam-to-column connections in fire conditions. The component method is now well-established as an analytical technique for rotational properties of connections at ambient temperature. In the context of the much higher rotations experienced at the ends of long-span beams in fire, together with high axial forces due to restrained thermal expansion, its justification changes. The importance of residual strength and stiffness of a connection is decreased, but it is essential that its ductility is represented properly in order to provide designers with the ability to match forces to strength at high temperatures.

1 INTRODUCTION

Research over the last decade has shown that composite floor structures can have a significantly greater fire resistance than is suggested by conventional tests on isolated elements. This is largely due to the interaction between the beams and floor slabs in the fire compartment, and the restraint afforded by the surrounding structure. This research is now being applied in the design of real projects, with an implicit assumption that, because they heat more slowly than the connecting members, the connections have sufficient fire resistance. However, observations from full-scale fire tests at Cardington and the collapse of buildings at the World Trade Centre in 2001 have raised concerns about this assumption. There is renewed interest in how connections respond to exposure to fire, and realistically this can only be investigated by examining complete structural assemblies, with suitable representation of the joints included. As structural fire engineering design increasingly optimizes the placement of protection materials in buildings, the axial forces generated in beams during the course of a fire, as revealed in non-linear three-dimensional analysis of large substructures, are seen to reach very high values. Typically these can change from compression in the early stages of a fire, when thermal expansion is resisted by surrounding structure, to tension in the later stages, when the heated members hang essentially in catenary. The connections at the ends of these members are therefore subjected in turn to these forces, whilst also being subjected to much larger rotations than are possible in ambient-temperature design.

The terminology on joints and connections has been standardized in EC3-1.8 (CEN, 2005b). A ‘connection’ is defined as the *location* where two or more members meet, and a ‘joint’ is defined as the *zone* where two or more members meet. This means that a ‘connection’ is considered as the parts which mechanically fasten the connected members; in the case of an end-plate connection these are the end-plate, the bolts, the welds and the column flange. A ‘joint’ is used as the more general term which includes the column web and the beam-end. For example, a beam-to-column joint can include two major-axis connections attached to the column flanges and two minor-axis connections attached to the column web. The principle is shown in Figure 1.

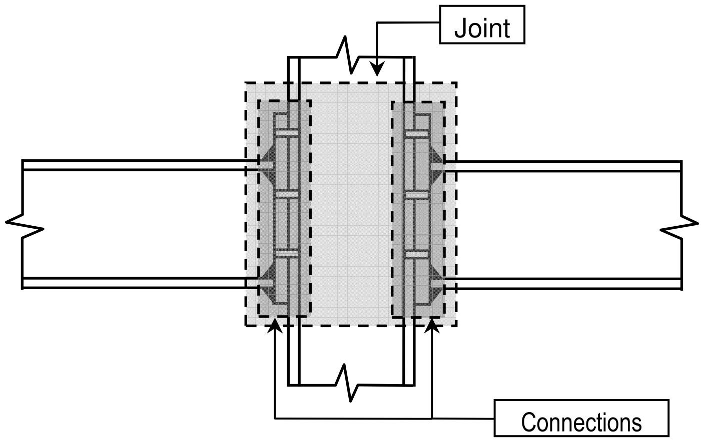

Figure 1: Joint and connection parts of a double-sided joint.

2 CONNECTION CHARACTERISATION

The main driving force behind the worldwide research effort over 30 years to represent connection properties has been a desire to achieve the advantages in design of buildings that can be derived from the real stiffness and strength of connections which would probably have been assumed as effectively pinned for normal design. The rotational stiffnesses inherent in normal simple connection details have the capability to reduce mid-span sagging moments in beams at the Ultimate Limit State, permitting sections with lower moment capacity to be used. Perhaps even more importantly, this rotational stiffness can reduce very significantly the deflections of beams for the Serviceability Limit State, which tends to control the selection of sections for long-span systems. It was this opportunity, afforded by the prospect of taking account of real connection characteristics in structural frame analysis, that stimulated a lengthy research effort to classify and quantify these characteristics.

An excellent account of the early work on connections is given by Nethercot and Zandonini (1989). In general, joints are defined in terms of rotational stiffness, strength and rotation capacity. The rotational stiffness of a joint is defined as the initial slope of its moment-rotation curve. In the 'pinned' case the rotational stiffness is zero and no rotational continuity exists between the beam and the column. The 'rigid' case allows no relative rotation between the beam and the column, and therefore the full beam-end moment is transferred. There are clearly cases which are close to the two extremes; a fully welded joint with column web stiffeners is almost rigid, and a web cleat connection with slotted holes is almost pinned. Nevertheless, the majority of practical joints are semi-rigid; some relative rotation occurs between the beam and the column, and a moment which depends on the relative rotational stiffnesses of the connection and the connected members is transferred. To simplify design, EC3-1.8 specifies boundaries between joint classifications, as shown in Fig. 2.

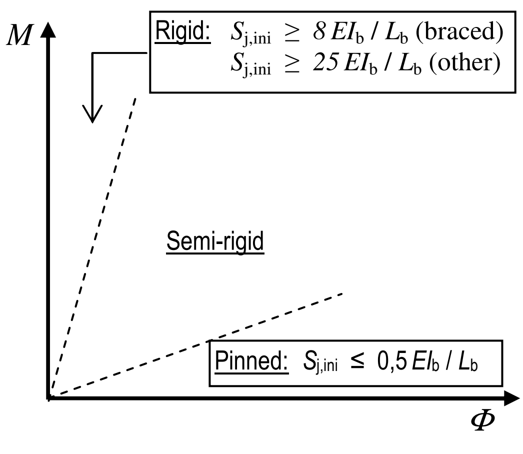

Figure 2: Stiffness classification of joints after EC3-1.8.

The EC3-1.8 strength classification compares the resistance moment of the connection with the moment capacities of the connected members. If the bending resistance of the joint is larger than the plastic moments of the members then it is 'full-strength'. If the joint bending resistance is less than 25% of that of one of the members and has sufficient rotational capacity it is nominally 'pinned'. Between these boundaries joints are 'partial-strength'. Most practical end-plate connections are in this category, which means that, if plastic design is used the plastic hinges will form in the joints and not in the adjacent members. Hence, sufficient rotational capacity is required in the joint to form a plastic mechanism and develop a full plastic moment at the mid-span of the beam.

The ductility of a joint is its ability to maintain its plastic moment over a sufficient rotation to develop a plastic mechanism in the adjacent members. A joint which fully achieves this is Class 1: 'ductile' (Jaspart, 2000). The lower bound of the ductility classification is Class 3: 'brittle', and may only be used in elastic frame design. Class 2: 'semi-ductile' lies between these extremes. The boundaries between the classes are not defined generally in EC3-1.8, and ductility of joints is treated in a very approximate manner, reflecting the sparse research in the field prior to its publication. Only been in recent years have researchers focused on the available ductility of a joint. Simões da Silva and Girão Coelho (2001a), Kühnemund (2003), Girão Coelho (2004), Beg *et al.* (2004) and Girão Coelho *et al.* (2005) all have used the Component Method, which is described below, to predict ductility for semi-rigid joints.

In general, five different ways of representing the moment-rotation response of semi-rigid joints exist:

1. Mathematical expressions as curve-fit models,
2. Simplified analytical models,
3. Mechanical (spring) models,
4. Finite element models,
5. Macro-element models.

Macro-element models are a relatively recent development. These combine mechanical and finite element principles, as they use finite element formulations to incorporate mechanical or curve-fit models into frame analysis. This type of modelling is particularly suitable for elevated-temperature analysis.

2.1 *Curve-fit models*

These are mathematical expressions fitted to moment-rotation curves found in experiments. The expressions include linear, bi-linear, tri-linear and polynomial, power series and B-spline functions. These are described in detail by Jones *et al.* (1983) and Nethercot and Zandonini (1989). A model which has been used at ambient and elevated temperatures uses the so-called Ramberg-Osgood (1943) curve, modified by Ang and Morris (1984) to represent the moment-rotation curves of joints, and extended by El-Rimawi (1989) to elevated temperatures. The approach is shown in Equation 1:

$$\Phi_c = \frac{M_c}{A} + 0.01\left(\frac{M_c}{B}\right)^n \tag{1}$$

where Φ_c is the joint rotation and M_c is the corresponding moment. This expression has been used by Leston-Jones (1997) and Al-Jabri (1999) to model their elevated-temperature test data for bare steel and composite joints. It can be applied to fire cases by making the terms A and B temperature-dependent. These factors control the stiffness and capacity of the joint respectively, whereas the index n controls the shape of the moment-rotation curve.

Although curve-fit models of joints are very easily integrated into frame analysis as rotational springs, they can only be used for joints which have been subject to testing. For high-temperature cases, axial forces acting on the connection cannot be represented easily in this approach unless a wide range of combinations of moment, rotation, axial force and temperature have been tested. This is impracticable due to the high cost of experiments and the vast number of connection configurations used in practice.

2.2 *Component modelling*

A more practical approach is the use of mechanical models, particularly the so-called "Component Method". This method was initially developed by Tschemmernegg *et al.* (1987) for ambient temperature conditions and after much development is now included in EC3-1.8. The principle is to consider a joint as a set of basic zones, each of which performs an individual structural action. These zones can be considered as assemblies of non-linear springs whose combination forms a model for the whole joint. The principal component zones of an end plate connection are shown in Fig. 3.

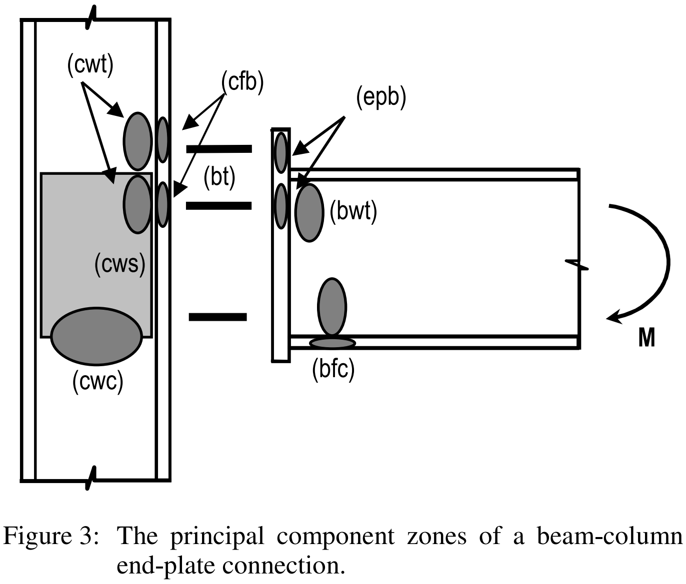

Figure 3: The principal component zones of a beam-column end-plate connection.

3 AMBIENT-TEMPERATURE COMPONENT MODELLING OF JOINTS

There are a number of different options (elasto-plastic, bi-linear, multi-linear or non-linear) to approximate the behaviour of the component springs. In EC3-1.8 each component is characterised by an initial stiffness k and a design resistance F_{Rd}, which are linked in an elastic-perfectly plastic fashion. This simple approximation allows a direct calculation of the moment-rotation curve of the joint. For better accuracy in the joint approximation more complex force-displacement models can be used, derived from test results, finite element models or preferably from simplified mechanical models. This increase in complexity of the component representation makes it necessary to solve the final spring model iteratively, which is not a problem if the spring model is incorporated into a non-linear finite element program. Although the Component Method associates each component with a certain internal force in the joint, in reality some components are exposed to stresses in more than one sense. EC3-1.8 specifies reduction factors, for the presence of shear (ω) and longitudinal stress (k_{wc}) in the column web in compression, and for the presence of shear stress (ω) in the column web in tension.

The Component Method can be extended at this stage to elevated temperatures by using high-temperature material properties with the ambient-temperature component models. This principle has been used by Leston-Jones, Al-Jabri and Simões da Silva *et al.* (2001b). The alternative, of developing new multi-linear elevated-temperature components models, has been adopted by Spyrou (2002, 2004a, 2004b).

The final step of the Component Method is to assemble the components and determine the resulting moment-rotation curve. Each component is represented as a translational spring interconnected by rigid links. The spring model for the joint of Fig. 3 is shown in Fig. 4 below.

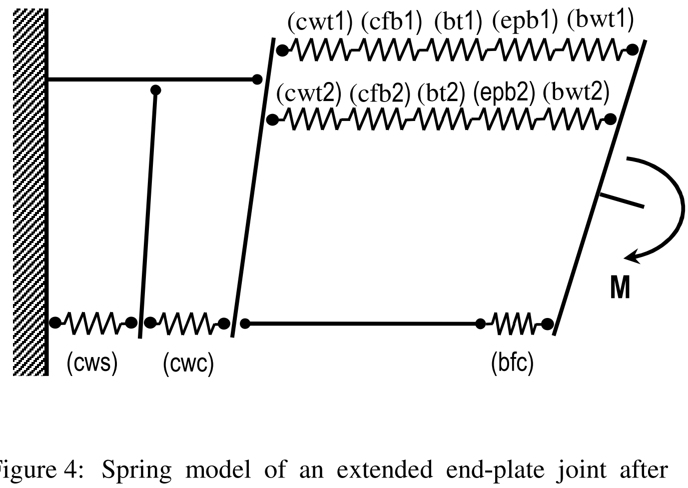

Figure 4: Spring model of an extended end-plate joint after EC3-1.8.

The moment resistance $M_{j,Rd}$, of a joint is:

$$M_{j,Rd} = \sum_{i=1}^{n} F_{ti,Rd} \, z_i \tag{2}$$

where $F_{ti,Rd}$ is the design tension resistance of bolt row i, and z_i is the distance from bolt row i to the centre of compression. The resistance of each bolt row $F_{ti,Rd}$ is that of the weakest component in this row, or either of the resistance of the two components in compression or the shear panel. However, this equation is only effective for a bolt row if the distance to the next row is sufficiently large, and the column flange and the end-plate develop individual failure mechanisms. If this is not the case two or more bolt rows may fail as a group and the resistance is lower than the sum the individual rows.

To calculate the rotational stiffness of the joint, the complete spring model shown in Fig. 4 can be simplified, replacing each bolt row by an equivalent spring of stiffness

$$k_{et,i} = \frac{1}{\dfrac{1}{k_{cwt,i}} + \dfrac{1}{k_{cfb,i}} + \dfrac{1}{k_{bt,i}} + \dfrac{1}{k_{epb,i}} + \dfrac{1}{k_{bwt,i}}} \tag{3}$$

where the stiffness of the beam web in tension k_{bwt} is assumed to be infinite. The compression and shear components can similarly be represented by equivalent springs. This simplifies the spring model in Fig. 4 to that shown in Figure 5(a). The equivalent spring model can be simplified even further, so the springs for each bolt row can be replaced with a single equivalent spring, as shown in Figure (b). Having calculated the moment resistance and initial rotational stiffness of the joint, EC3-1.8 offers two options, one bilinear and one curvilinear, to approximate the moment-rotation curve. These moment-rotation curves can then be introduced as rotational springs at beam ends into frame analysis programs.

It is apparent that the calculation process of the Component Method is quite lengthy, and therefore a number of programs have been developed to simplify its application in engineering practice. The software CoP, developed at the University of Liège and RWTH Aachen, is a current example of such software.

4 EXPERIMENTS ON JOINTS AT ELEVATED TEMPERATURES

4.1 Moment-rotation-temperature testing

The first experimental fire tests on joints were conducted by Kruppa (1976) at CTICM, on six joint types ranging from "flexible" to "rigid". Their primary purpose was to investigate the performance of high-strength bolts at elevated temperatures, and no indication of the performance of the joints was reported. Two tests were carried out by British Steel (1982) on a "rigid" joint. Despite the limited scope, it was concluded that joint elements could suffer significant deformation in a fire. Lawson (1989, 1990) was the first to measure the rotations of 8 cruciform joints with different major-axis connections exposed to the Standard Fire, at constant load. Five of his tests were on non-composite beams, two on composite beams and one on a shelf-angle floor beam. Of the steel joints 3 types of typical joints were studied (extended and flush end-plates and a double sided web cleat). The tests showed that up to two thirds of the ambient temperature design moment capacity could be sustained in standard fire conditions. It was noted that the bolts did not fail prematurely, and rotations always exceeded 6°. It was clear that composite action in fire enhanced the moment capacity of the joints, which could be estimated by superposing the capacities due to the bare-steel joint and the slab reinforcement. Lawson proposed simple design rules (later withdrawn) based on BS5950 Part 8 (BSI, 2003) for designing simply supported beams in fire taking into account the joint moments. Although these tests results provided insufficient data to describe full moment-rotation-temperature characteristics of the joints, they did provide essential information for early attempts at joint modelling.

The first systematic series of high-temperature tests producing moment-rotation curves at different

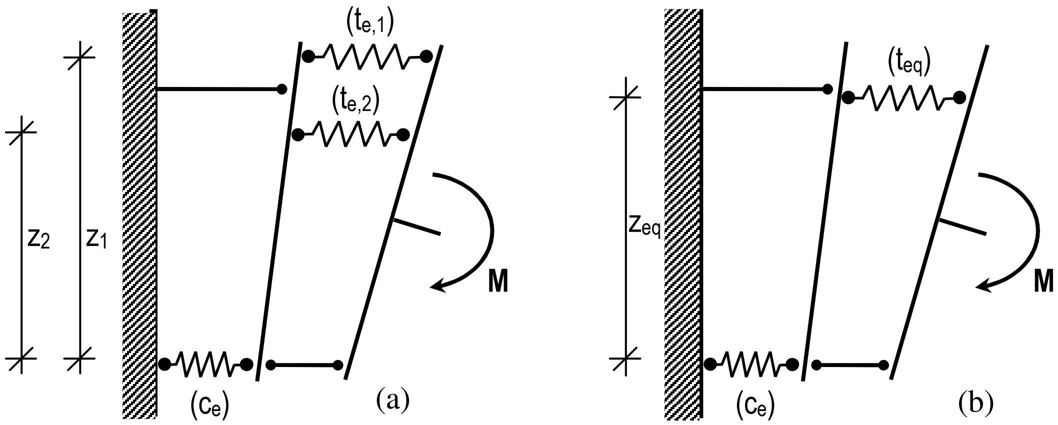

Figure 5: Equivalent (a) and simplified (b) spring models after EC3-1.8.

temperatures were conducted by Leston-Jones (1997) in work done at the University of Sheffield and BRE. Eleven tests were carried out on flush end-plate joints, both bare-steel and composite, using small standard sections, including two tests at ambient temperature. Both stiffness and moment capacity decreased with increasing temperature, particularly in the range 500-600°C. These tests provide useful data for connection modelling, although their range of details is very limited, and allow verification against earlier ambient-temperature tests by Davison *et al.* (1987). Continuing Leston-Jones's work at a time when the data from the full-scale tests at Cardington was becoming available, Al-Jabri (1999) extended the scope of this test programme to study the influence of parameters such as member size, connection type and different failure mechanisms. In total 20 tests were conducted on flush end-plates with different section sizes and on flexible end-plate joints, both bare-steel and composite. Particular attention was given to joint details used in the composite building at Cardington. Fig. 6 shows the furnace and experimental fire testing set-up used by both Leston-Jones and Al-Jabri.

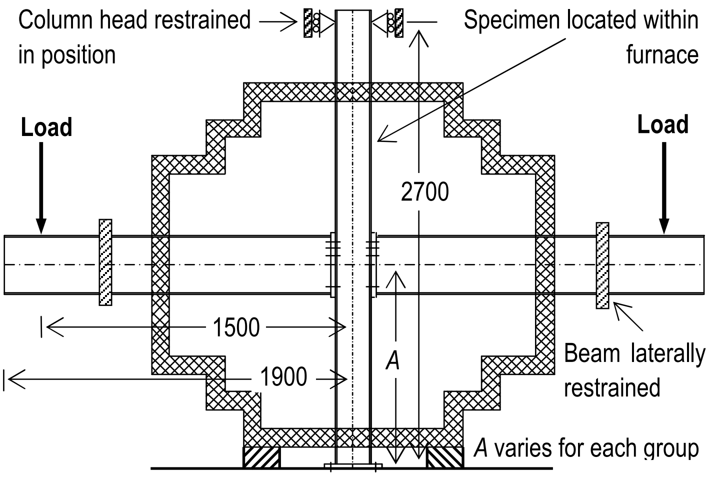

Figure 6: Cruciform test scheme used by Leston-Jones (1997) and Al-Jabri (1999).

During the course of this project data and observations from the Cardington full-scale frame fire tests made it apparent that moment-rotation tests on isolated joints are not sufficient to describe the behaviour of joints and connections in framed buildings in fire. The very high axial forces in the beams caused by the restraint to thermal expansion had clearly had a major influence, which was particularly evident from the extent of local buckling near the beam ends.

Some moment-rotation testing in fire has continued since this time, largely in order to validate nonlinear FE modelling approaches. Two axially unrestrained cruciform joints with extended end-plate connections, using relatively large beam and column sections, have recently been tested in China by Lou and Li (2006). Whereas these connections failed by buckling of the column web at ambient temperature, the failure mode changed at elevated temperatures to fracture of the bolts and yielding of the column web in tension, even though the end-plate temperatures close to the bolts were lower than the column web temperature.

4.2 Restrained high-temperature joint testing

Despite the evident importance of modelling the behaviour of unprotected joints in a restrained condition at high temperatures, no experimental studies concerned with this matter have yet been published in the open literature at the time of writing.

In a joint project of the Universities of Manchester (Liu *et al.* 2002) and Sheffield (Allam, 2003), some loaded furnace tests on restrained beams supported by columns creating 'rugby-post' frames were conducted. The columns and connections were fire-protected, and remained at relatively low temperatures, and the main aim was to investigate the effects of translational and rotational restraint to the beam. High axial compressive forces were recorded early in the fire, but as the vertical beam deflections increased these progressively changed into tension (catenary force) and increased the failure temperature of the beam considerably compared with the unrestrained condition. Although, the connections were protected the tests gave practical evidence of the axial forces acting on a connection at elevated temperatures following the standard fire curve. In retrospect it is unfortunate that no information on the forces in the beam or connections was recorded in the cooling phase.

A hitherto unpublished experimental series of six internal extended end-plate joints has been tested at temperatures between 400°C and 700°C by Qian (2006) and Tan in Singapore. These tests were designed to examine particularly the shear panel at the end of the connected beam. The first three tests were conducted at 700°C with different actively-applied axial force in the beams. These tests failed in combinations of end-plate bending and shear deformation of the shear panel. In the remaining three tests, the end-plate thickness was increased to 40mm in order to isolate the shear panels. These tests were conducted at 400°C, 550°C and 700°C, and the capacity of the shear panels reduced as expected with increasing temperature. This test data is currently in press, and will provide an opportunity to validate the different approaches to modelling steel connections in fire, including the effects of restraint.

5 HIGH-TEMPERATURE COMPONENT TESTING

Spyrou *et al.* (2004a) conducted an experimental investigation of the performance of the tension and compression zones of steel joints at elevated temperatures. These steady-state tests included 45 T-stubs in tension and 29 column web transverse compression tests, most of them at elevated temperatures. Simplified mechanical models of both the tension and compression zones were developed and compared with the experimental results. The analytical model for the tension T-stubs proved capable of predicting with reasonable accuracy the failure in any one of the three classic modes (Fig. 7):

1. Formation of plastic hinges in the flange near the web followed by bolt yield and fracture,
2. Formation of plastic hinges in the flanges near the web and the bolt lines followed by bolt yield and fracture,
3. Bolt yield and fracture with the flanges remaining elastic.

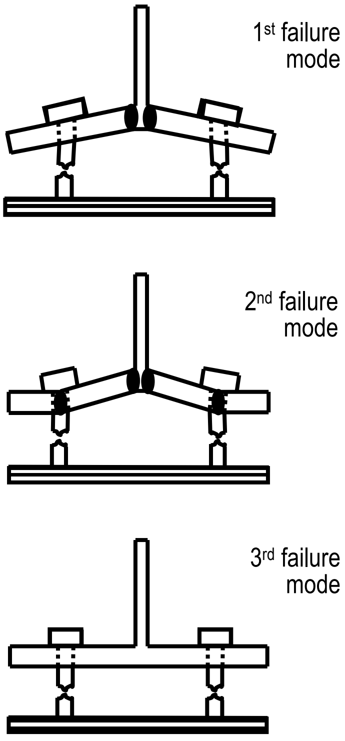

Figure 7: The three failure modes of a T-stub in tension.

Spyrou (2004b) also examined the main component of the compression zone for major-axis flush end-plate joints, which is the column web, compressed by the lower beam flange through the end-plate and the column flange. He tested 29 such arrangements under steady-state temperature conditions, using the furnace setup shown in Fig. 8.

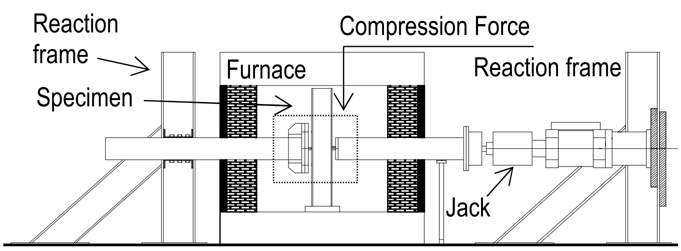

Figure 8: Arrangement for compression zone tests.

Spyrou developed a semi-empirical mechanical model for ultimate compressive force after comparing a large number of previously published equations with his tests at elevated temperatures. Eventually he adapted an ambient-temperature equation by Drdacky (1977), originally derived for thick plate girder webs, to the form

$$P_u = t_{wc}^2 \sqrt{E_{wc}\sigma_{wc}} \sqrt{\frac{t_{fb}}{t_{wc}}} \left\{ 0.65 + \left[\left(\frac{1.6c}{d_{wc}} \right) \left(\frac{2\beta}{2\beta + c} \right) \right] \right\} \tag{4}$$

where β is defined in Fig. 9. The EC3-1.2 strength and stiffness reduction factors are used to reduce the yield strength and Young's modulus of the web material.

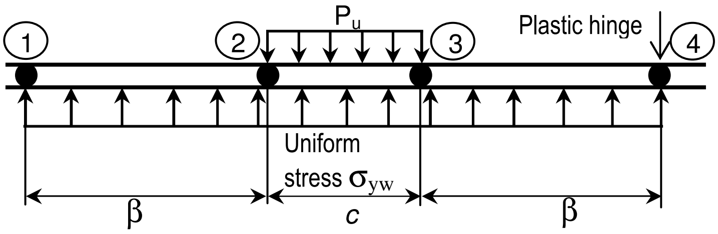

Figure 9: Assumed mechanism of web yielding.

An empirical model for the curve prior to the ultimate state was derived based on experimental observations, together with 2-D and 3-D finite element analyses. The results from these finite element analyses and the simplified model compared very well with the test results, over the whole range of tests. The clear logic of the comparison was that the load capacity of the compressed web is essentially controlled by the development of plasticity in the web-plate, and that inelastic buckling is essentially a secondary effect. This was repeated across the whole range of web slenderness tested, as well as for some more slender webs analysed using ANSYS.

Block (2005a, 2005b) extended Spyrou's work on the column web compression zone, to take account of the possible effect of superstructure loading in the column on the resistance of the web-plate to horizontal patch compression. Kuhlmann and Kühnemund (2000) have shown that the compression zone in the column web is critical if rotational capacity is needed, especially in composite connections. It is even more important in fire because of the large compressive forces which can be induced in the column web due to thermal expansion of the beams. They also showed that the characteristics of this zone can be significantly affected by the normal axial load in the column interacting with the transverse loads on the column web induced by beam effects. The present work therefore extends Spyrou's approach by developing a suitable representation for this part of the joint for inclusion in the overall spring model.

The simplified model for predicting the force-displacement behaviour of the compression zone at elevated temperatures is defined by three aspects – its ultimate resistance, the corresponding displacement, and a suitable force displacement relationship. In the present formulation, the ultimate resistance is based on a proposal by Lagerqvist and Johansson (1996). This assumes a series of plastic hinges forming in the column flange in combination with yielding of the web, with a reduction to account for buckling in slender webs. The displacement at ultimate load takes the form of an empirical equation which has been fitted to a large number of finite element models, varying geometrical and material parameters. Finally the force-displacement relationship is obtained by a curve-fitting approach using the initial stiffness approach given in the EC3-1.8 and a Ramberg-Osgood type of equation. The effects of elevated temperatures are accounted for by using the temperature reduction factors for steel given in EC3-1.2.

The reduction factor proposed by Kuhlmann and Kühnemund to account for axial forces in the column has been included, again with a modification to allow for elevated-temperature conditions. The ultimate strength of the column web under a combination of transverse and axial loads is then given as a function of temperature.

6 THE COMPONENT METHOD IN FIRE

In this paper two different component-based approaches are taken to modelling the behaviour of end-plate joints at elevated temperatures. These approaches are each quite valid, but represent different levels of possible analysis for performance-based design.

6.1 Modified Rotational Model.

A logical way of adapting the ambient-temperature Component Method's calculation of joint rotational stiffnesses to high-temperature conditions has been used by Simões da Silva *et al.* (2001b), and by Al-Jabri *et al.* (2005). This is to apply the ambient-temperature component models given in EC3 Part 1.8, using the material reduction factors given in EC3 Part 1.2 for stiffness, limit of proportionality and yield to amend the mechanical characteristics of the main components at high temperatures. This method focuses on establishing elevated-temperature rotational characteristics for end-plate beam-to-column steel joints, but does not directly include the normal stiffness or strength of the connection in the direction of the beam axis, and therefore does not allow the connection to contribute its flexibility or ductility to the estimation of beam forces. Nevertheless, it is capable of offering a practical and economical way of introducing rotational joint behaviour into whole-frame modelling in fire, while giving upper-bound solutions for the joint's tying force.

The moment capacity and rotational stiffness of a joint are calculated according to the principles set out in equations (2) and (3). The individual component stiffnesses and strengths may be derived either from research studies such as those by Spyrou and Block, based either on testing or on non-linear numerical analysis. Alternatively the calculation procedures and equations set out in EC3-1.8 for these characteristics at ambient temperature may be used, provided that the Young's modulus and yield strength are multiplied by the appropriate EC3-1.2 reduction factors $k_{E,\theta}$ and $k_{y,\theta}$ for elevated temperatures. This was the method employed by Simões da Silva *et al.* (2001b). This method was compared with the results from the testing by Al-Jabri (1999), with very reasonable correlation being obtained, even under the restriction of an isothermal assumption. An advantage of this method is that, at least for end-plate connections, it is possible to take advantage of the results of 20 to 30 years' research which has culminated in the component models provided in EC3-1.8. Its principal disadvantage is that it is aimed at producing only the rotational characteristics of joints.

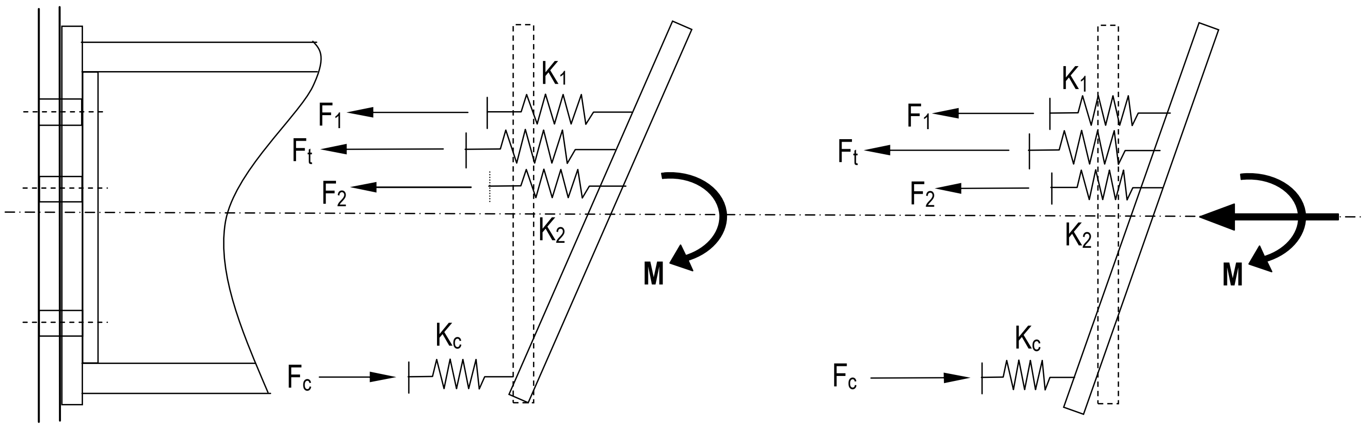

Figure 10: Component model under pure moment and axial force combined with moment.

6.2 General Connection Element.

The second method is the more general one, of assembling the components into a connection element and using this within the numerical analysis software directly. In this case no "rotational stiffness" is assigned to the connection, but any component for which characteristics have been defined will mobilise its own displacement degree of freedom as part of the overall equilibrium of the joint.

In the context of whole-structure modelling in fire, the justification for using a component-based approach is more compelling than simply obtaining reductions of rotational charactersitics. Structural members will undergo considerable thermal expansion in fire, with strains much greater than normal yield strain levels for typical structural materials such as steel. For exposed beams these expansions will be resisted by restraint from columns, bracing, building cores, attached slabs and adjacent structure. In addition to this effect, the material weakens progressively as it heats, and thus the beam loses its capacity to resist its loads in bending. Thus beams may be subjected, at different times during a fire, to:

- High compressive forces as heated beams expand against restraint.
- Tensile forces close to the high-temperature capacity when bending action has been supplanted as the main load-carrying mechanism by catenary tension at very high temperatures.
- Very high tensile forces as a member shrinks and stiffens simultaneously during cooling.

The joints connecting the members to columns are subjected to these high normal forces, in addition to the vertical shear (for which they have probably been designed at ambient temperature) and rotation. It is therefore of limited use to attempt to model the connections as part of a larger structure in fire conditions on the basis of moment-rotation-temperature characteristics alone. Clearly it is impractical to establish databases with full variation of moment, rotation, temperature, normal force and normal deflection for a range of typical connections. However, if a connection is modelled as its appropriate assembly of components, each with an established nonlinear temperature-dependent axial force-deflection characteristic, this assembly can simply be placed at the beam-end, connecting it to the column face, and the mathematical linkage between the rotational and normal degrees of freedom of the connection is replaced by the compatibility conditions between the beam-end and the column-face. This reduces the problem to one of establishing, either by modelling or testing, the normal force-deflection characteristics at different temperatures for the relatively simple component springs.

This was the rationale which underpinned the work by Spyrou *et al.* (2004[a], 2004[b]) in developing simplified force-deflection-temperature models for the tension and compression zones of end-plate connections. He was able to demonstrate good correlation with previous elevated-temperature moment-rotation tests, but did not formulate a full connection element, and of course at the time no test data was available which included a net normal force on the column through the connection. Other researchers (Vimonsatit *et al.* 2006[b], Tan *et al.* 2006) have developed complementary analytical descriptions for primary mechanical components, so that an adequate basis now exists for mechanical modelling of steel beam-to-column end-plate joints, although there is room for refinement of the component models.

Block (2006) has constructed a general component-based connection element to be used in global high-temperature frame analysis, using the basic spring model shown in Fig. 11.

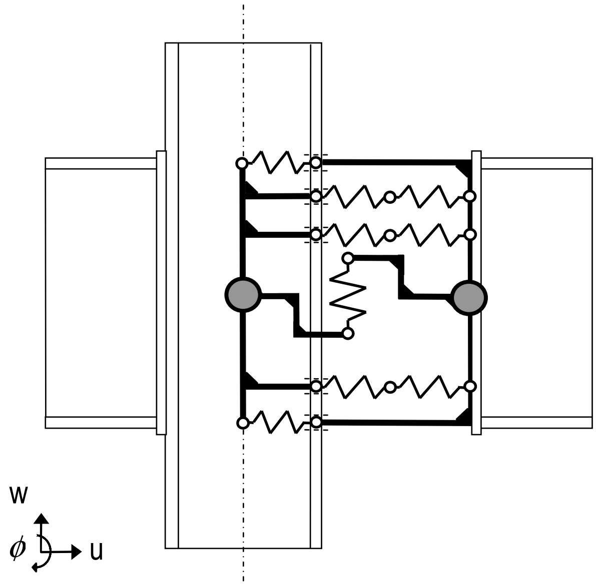

Figure 11: Spring model representing a typical joint.

This was developed from a simpler model by Spyrou (2002) to represent a real end plate joint as a number of discrete components: endplate in bending, column flange in bending, bolts in tension, and column web in compression. The first three components form the tension zone of the connection and are combined as two T-stubs in series. An additional vertical shear spring, currently assumed to be rigid in the absence of the necessary studies to produce a component model, is included in order to transfer the vertical load from one node to another, leaving the vertical and horizontal stiffnesses of the element uncoupled.

An alternative to this approach would be to use a diagonal component spring in a beam end-panel of finite length, and since this is a direct analogy to the tension field which carries shear to the connection this has some attractions. Qian (2007) has investigated this shear panel experimentally, over specific ranges of web slenderness, but at the present time this work has not yet produced a working model for the beam-end shear component.

7 CONCLUSION

Some experimental validation of component methods in fire conditions has been done by Simões da Silva (2001b), Al-Jabri (2005) and Block *et al.* (2006). A typical comparison, in this case by Block, using the general connection element with Leston-Jones's tests is shown in Fig. 12.

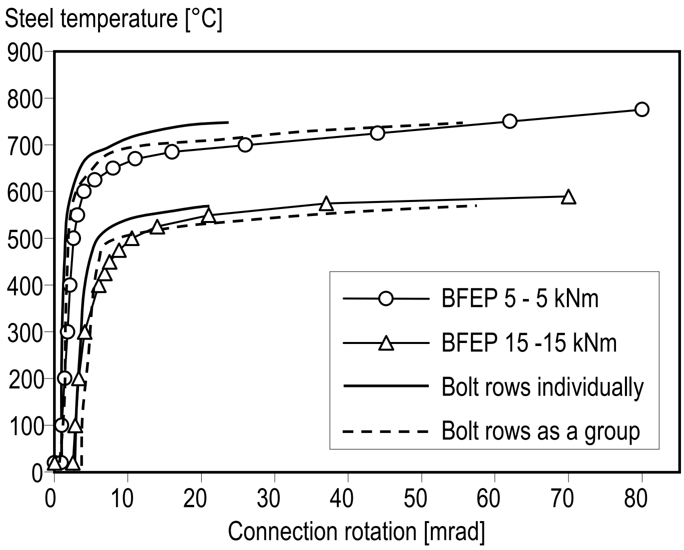

Figure 12: Comparison of the element with high-temperature tests BFEP 5 and BFEP 15 by Leston-Jones.

Although similar comparisons have been done by others, in common with Block's comparison they suffer from the nature of the test evidence itself. As has been stated previously, there is currently very little data from furnace tests which include normal force on the connection zone; this is clearly an extremely complex and potentially expensive form of testing which is beyond the normal capabilities of test furnaces. The tests by Liu *et al.* (2002) on "rugby-post" frames were not really intended to impose restraint conditions on the connections themselves but on the beam members, and so the joints were protected in these tests. There is a clear need for properly instrumented testing of connections under the full range of combinations of moment and normal force which would be experienced in framed structures. The use of restraint to thermal expansion in loaded frames such as Liu's gives too limited a range of combinations for validation of the component-based models, because it is unlikely ever to be feasible to test such frames of the range of spans which are common in contemporary steel and composite buildings.

Under many circumstances it is likely that both the types of component modelling presented above will produce very similar results when used in structural modelling. This will not be the case when connections are relatively ductile, or when the normal force (in either direction) dominates the connection moment. This is especially important in full-structure or extended substructure analysis, when connection ductility in the sense normal to the column can allow enough beam deflection to reduce the joint force to a level which can be sustained by the connection. If this ductility is not present beam forces in catenary action can be extremely high. The full component-based connection model therefore allows the designer to amend the connection detail so that the required ductility is present without causing fracture of parts of the connection.

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