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PROGRESSIVE FAILURE MODELLING AND DUCTILITY DEMAND OF STEEL BEAM-TO-COLUMN CONNECTIONS IN FIRE

Ruirui Sun[1], Ian W Burgess[2], Zhaohui Huang[3], Gang Dong[4]

ABSTRACT

A numerical procedure has been developed to model the sequences of failure which can occur within steel beam-to-column connections under fire conditions. In this procedure two recent developments, a static-dynamic solution process and a general component-based connection element, have been combined within the software Vulcan in order to track the sequence of local failures which lead to structural progressive collapse in fire. In particular it can be used to investigate how the structural behaviour, particularly the ductility and fracture of parts of the steel-to-steel connections, influences the progressive collapse resistance of steel frames in fire.

In the component-based connection element, a connection is modelled as an assembly of “bolt-rows” composed of components representing different zones of mechanical behaviour whose stiffness, strength, ductility and fracture under changing temperatures can be adequately represented for global modelling. The potential numerical instabilities induced by fractures of individual components can be overcome by the use of alternate static and dynamic analyses. The transfer of data between the static and dynamic analyses allows a seamless alternation between these two procedures to take place. Accuracy and stability of the calculations can be ensured in the dynamic phase, provided that the time steps are set sufficiently small. This procedure has the capacity of tracking the sequence of local failures (fractures of connection components, detachment and motion of disengaging beams, etc.) which lead to final collapse.

Following an illustrative case study of a two-bay by two-storey frame, the effect of ductility of connections on the collapse resistance of steel frames in fire is demonstrated in two case studies of a generic multi-storey frame. It is shown that the analytical process is an effective tool in tackling the numerical problems associated with the complex structural interactions and discontinuous failures which can affect a steel or composite frame in fire, potentially leading to progressive collapse. It can be seen that both tensile and compressive ductility in the connections make a contribution to the fire resistance of the beams. Preventing the detachment of steel beams in fire can be achieved by inducing greater ductility into their connections. Combined with appropriate component-based connection models, this procedure can be adopted in performance-based fire-resistant design to assess the ductility requirements of steel connections.

KEYWORDS: Progressive collapse, Steel connections, Ductility demand, Fire, Static-dynamic analysis, Component-based modelling.

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[1] Engineer, MMI Engineering Ltd, Aberdeen, AB12 3JG, Email: sr_1123@hotmail.com

[2] Professor, Department of Civil and Structural Engineering, the University of Sheffield, Sheffield, S1 3JD, UK, Email: ian.burgess@sheffield.ac.uk

[3] Reader, School of Engineering and Design, Brunel University, London, UB8 3PH, UK, Email: zhaohui.huang@brunel.ac.uk

[4] Engineer, MMI Engineering Ltd, Warrington, WA4 6HL, Email: GDong@mmiengineering.com

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1 Introduction

Progressive collapse of a structure occurs when an initial local failure causes a sequence of failures in other elements, eventually resulting in collapse of a disproportionately large part of the structure, or even the entire building. It has been an important structural issue in the UK since the collapse of a large part of the Ronan Point apartment building in London in 1968 [1]. In steel-framed buildings connections must provide structural continuity, redundancy and ductility. Failure of connections can cause structural discontinuities and reduce the resistance provided by alternative load paths. An accurate understanding of connection performance is therefore essential to the assessment of structural robustness, and therefore to structural design against progressive collapse.

The collapse of World Trade Center towers [2] has attracted attention to the robustness of steel structures in fire, within which the need to evaluate the performance of the steel connections at elevated temperature has become a key topic. The fire tests at Cardington [3], and the collapse of at least one of the WTC buildings [4], illustrated that the connections are potentially the most vulnerable parts of a structure in fire. It is traditionally assumed that beam-to-column connections have sufficient fire resistance, because of their lower temperatures and slower rate of heating than the members to which they are attached. However, the full-scale fire tests at Cardington in the 1990s showed that the connections are actually more vulnerable than previously assumed during fire, because the forces and deformations to which they are subjected during a fire differ significantly from those assumed in general design, either for the ambient-temperature Ultimate Limit State or for the Fire Limit State. The internal forces in connections change in three stages, starting with moment and shear at ambient temperature, and becoming moment, shear and compression due to thermal restraint to beam expansion in the initial stages of a fire, and finally changing to shear and tension at high temperatures, when beams essentially hang in catenary. If a connection does not have sufficient resistance or sufficient ductility to accommodate the large rotations and normal forces which it experiences at any stage of fire exposure, connection fracture will occur, and this may lead to extensive damage or progressive failure of the structure [5, 6].

These complexities are difficult to apply in design practice. Full-scale structural testing is expensive, and its reliability is considerably affected by the particular test methods and laboratory facilities used. Hence, assessment of the robustness of steel connections in fire inevitably relies on numerical modelling using appropriate predictive methods.

The component method is a practical approach to the modelling of connection behaviour. In the method which was initially proposed by Tschemmernegg and Humer [7], the connection is represented by an assembly of individual components with known mechanical properties, and it has been validated as an adequate, if not 100% accurate, representation of the key behaviour of certain connection types in fire. Leston-Jones [8] applied this method to his cruciform tests as a way of representing rotational properties at elevated temperatures; Al-Jabri [9] used the method to model the rotational behavior of flexible endplate connections measured in his high-temperature experiments. Spyrou et al. [10] conducted a large number of high-temperature component tests, and combined these components by means of a simple two-spring connection model which, for the first time, was capable of modelling simultaneous rotational and axial deformations and forces, subject to the temperature-dependent reduction factors on material properties which were at the time given in the ENV version of Eurocode 3 Part 1-2 [11]. Block et al. [12, 13, 14] developed Spyrou's approach to be used in practical modelling of the in-plane behaviour of connections between steel structural elements as part of global 3-dimensional analysis of framed structures subject to heating by design fire scenarios. Simões da Silva et al. [15] employed the component properties established in the published version of Eurocode 3 Part 1-8 [16] for ambient-temperature semi-rigid frame design, together with EC3 material reduction factors, to model Leston-Jones's rotational cruciform tests at high temperatures. Ramli Sulong et al. [17] also included high-temperature connection behaviour in the finite element program ADAPTIC using the component method.

However, up to the present day only rather limited studies have focused on progressive failures (fractures), and the impact of ductility of connection components, on the behavior of whole frames in fire. Huang et al. [18] carried out a series of tests under combined force components at elevated temperatures, on connections between steel beams and H-section columns, including reverse-channel and flush endplate connections. It was found that reverse-channel connections provide not only high strength, but also the high ductility which is required to reduce the possibility of connection fracture and thus improve the robustness of buildings in fire. Dong [19] developed a general component-based connection model into which appropriate component characteristics, such as those required to represent reverse channels or flush endplates, can be inserted. This development has the potential to enable the full performance of more types of connection, including the fracture of their components, to be integrated into global nonlinear structural fire analysis, initially implementing this in the Vulcan software. Burgess et al. [20] have given an overview of the sequence of research on connection behaviour in a fire at the University of Sheffield, emphasizing that connections are critical structural elements of building frames whose fracture can cause the collapse of the connected beams, possibly leading to progressive collapse of the building. Fang et al. [21] have proposed practical frameworks for the assessment of structural robustness in fire. Two alternative approaches are included; a temperature-dependent approach (TDA) and a temperature-independent approach (TIA). System failure is based on comparing ductility demand and ductility supply in the joints, in which the component-based method is used to characterize the non-linear connection response. The structural failure criterion which is adopted in this approach is associated with the first failure of a joint, even though it is possible for a structure to survive a sequence of component failures before a serious collapse occurs.

Fire causes relatively slow growth and decay of structure temperatures in buildings, compared with their key dynamic characteristics such as the periods of natural frequencies, so the evaluation of thermo-structural behaviour is conventionally carried out by static or quasi-static numerical analysis. Because of the nature of quasi-static analysis, it can usually only track a structure's behaviour up to the point at which its first component fails; in limited cases this may extend to more than one component's failure. In reality a connection may either be able to regain its stability after the initial fracture of one (or a few) components, or the first failure may trigger a cascade of failures of other components, leading to complete detachment of the supported members. In considering these possibilities it should be recognized that the forces on a connection may be relieved considerably after an initial component failure, so a cascade of failures is not inevitable. These possibilities should be considered in performance-based design modelling, when a structure is being assessed for robustness. If connections are to be designed to avoid the possibility of members becoming completely detached, this numerical modelling must be capable of predicting the sequence of failures of components, rather than considering the first loss of stability as signifying building failure.

After a beam has completely detached from its connections, the load-paths within the remaining frame are changed, and the effective lengths of vertical support members may be changed by the loss of restraint from the detached beam. These effects are important for structural robustness assessment. In order to model them in a single analytical run, a static-dynamic procedure has been combined with the parallel development of general component-based connection finite elements. This combination allows the behavior of a structure to be tracked, through the initial fracture of a connection component, via successive failures at bolt-rows (which may occur in different connections), and complete detachment of connections. The analysis can also continue to track the movements of disengaging members and the load-sharing mechanisms between members which still maintain their integrity and stability within the remaining structure, until total collapse happens. In this paper, the procedure will be described in detail, and the importance of the ductility of the connections in enhancing structural robustness in fire will be demonstrated.

2 Development of numerical procedure

Vulcan is a finite-element software package which has been developed at the University of Sheffield [22-25] for two decades, specializing in structural fire analysis of steel and composite framed structures. Recently, a process known as static-dynamic progressive collapse analysis [26, 27] has been implemented in the Vulcan code. This procedure allows alternate static and dynamic analyses, attempting to use the advantages of both where appropriate. It has the capacity to deal with the temporary instabilities which occur during a structural progressive collapse. It is believed that, when combined with the parallel development of general component-based connection elements, this process should be able to track the progression of failures in connections as a fire develops, potentially leading to the final total collapse of the frame.

The analytical sequence, referring particularly to progressive fracture of connection parts rather than generalized instabilities, is shown in Fig. 1. The combination of the static and dynamic solution processes and the component-based connection model overcomes the numerical instabilities induced by fracturing of components within the connections. It can be seen from Fig. 1 that the static analysis terminates when a numerical instability occurs, and the subsequent structural behavior and load redistribution among the other parts of the structure, including the connection components, is tracked by the explicit dynamic procedure. The alternate use of the static and dynamic solvers is repeated in order to overcome temporary instabilities and to track the failure sequences within the connections.

The progressive failures within a connection may eventually cause the end of a beam to detach completely from the column. The kinetic movement of this beam end after its disengagement is then simulated by the dynamic analysis, and the beam can eventually be deleted from the structural model if the connections at both of its ends have broken. The features of the analytical sequence, shown in Fig. 1, are described in detail in this section.

2.1 Component-based connection model

In the component-based connection model, a joint is divided into a collection of key components, each of which represents a local zone whose behaviour can be characterized as that of a nonlinear spring. The disadvantage of the method is that it de-couples the behavior of these zones, which can in reality be part of a continuum. The essential properties of each zone, including the temperature-dependency of its material characteristics, its physical details and its unloading characteristics, are taken into account in assigning properties to its components.

Fig. 2 shows the layout of a typical endplate beam-to-column connection component assembly. The connection element has two nodes. Node 1 is located at the intersection between the beam and column reference axes. Node 2 is the end-node of the beam. The connection internally consists of several tension bolt rows and two compression spring rows [19]. A tension bolt row is characterized as an effective spring representing the properties of a group of springs, assembled in series and representing the endplate in bending, a pair of bolts in tension and the column flange in bending. Its maximum resistance is defined by that of the weakest spring in the series. A vertical shear spring, currently assumed to be rigid in the absence of the studies necessary to produce a component model, is included in order to transfer vertical shear force from one node to the other, leaving the vertical and horizontal stiffnesses of the element uncoupled. At the stage of full detachment of the beam from the connected column, the vertical component is assumed to have vanished.

Connections in fire are generally subject to extremely large deformations, which take them considerably out of the elastic range, so that the adopted component characteristics are not simply used to represent the initial stiffness and elastic limit, as they are in ambient-temperature semi-rigid frame design, but also to account for the ultimate resistance and failure ductility over a range of elevated temperatures. The generalized component-based connection elements have been validated against structural tests, and it has been shown that these elements are capable of giving an acceptably accurate prediction of the structural behaviour of the connections for which the component characteristics under fire conditions have been developed, including their stiffness, strength, and ductility.

2.2 Dynamic modeling of failure process in connection

The fracture of a component can initiate a cascade of failures within a connection, because the redistributed forces may exceed the capacity of other components, and in consequence failures can follow in rapid succession until complete detachment occurs. Of course, the redistribution of forces can also extend to other members and connections of the structure, either during the failure sequence of the initially failing connection or subsequently. The failure sequences within the structural model as a whole are tracked by the explicit dynamic analysis. A typical equation of motion is expressed in the usual generalized form, as:

$$\boldsymbol{M}\ddot{\boldsymbol{u}} + \boldsymbol{C}\dot{\boldsymbol{u}} + \boldsymbol{F}(\boldsymbol{u}) = \boldsymbol{Q}(t) \tag{1}$$

Where $\boldsymbol{M}$ is the mass matrix, $\boldsymbol{C}$ is the damping matrix, $\boldsymbol{F}(\boldsymbol{u})$ is the internal force vector and $\boldsymbol{Q}(t)$ is the external force vector; t, $\mathbf{u}$, $\dot{\mathbf{u}}$ and $\ddot{\mathbf{u}}$ are time, displacement, velocity and acceleration vectors, respectively. The central difference method is adopted for time integration. The mass and damping matrices are formed on the basis of the lumped-mass assumption, in which masses are only lumped at the nodes [26]. In these terms, the component-based connection element introduces stiffness between two co-located nodes without any associated mass. Introducing zero-mass elements risks the stability of the explicit dynamic solution process. However, in a sub-frame model, the nodes to which the connection element is attached normally have some mass contributed by adjacent beam or column elements. Another issue for the explicit dynamic analysis is the critical time step (also known as the "stable limit"). It is crucial to keep the time increments less than the critical time step in order to maintain the stability and the accuracy of the explicit time integration. Arbitrarily large time increments violate the basic principle of the explicit procedure and consequently generate divergent "run-away" errors. A conservative estimation process for the critical time step for explicit integration in the beam-column frame model has been adopted in the static-dynamic procedure. The time step is bounded on the basis of the maximum wave propagation speed within the elements; this is related to the element length and the wave propagation velocity within the materials.

2.3 Ductility within the connection

One of the motivations for developing the procedure is to facilitate the evaluation of structural robustness in fire, including consideration of the ductility of components within the connections, which has so far been neglected in the analysis for conventional fire resistance design. In recent studies [12, 17], the ductility limit of a connection has been conservatively based on the first fracture of a component within a connection. The successive failures and load redistribution within the connections also contribute to the robustness of the structure as a whole. The fracture of a component in tension is simulated by instantaneously allowing both its strength and stiffness to vanish. If any numerical instability occurs in such a case, the dynamic solution process is then initiated to allow the analysis to continue. If the force shed by the failed component can be redistributed among the other components of a connection in a stable fashion before complete detachment happens, the static analysis is then re-activated and the temperatures are allowed to change further.

The progressive failure of components in a connection, as both temperature and displacement change, can eventually induce complete detachment of the beam. It should be noted in this respect that thermal contraction due to cooling may be just as damaging as material weakening due to heating. The load path from the beam to the previously connected column is lost, and the beam starts to fall down, together with any attached débris. It should be noted that the impact of this débris may trigger progressive collapse in the lower floors, although this effect is out of the scope of the present study. This kinetic motion of the disengaging beam can be simulated by the dynamic analysis. When the deflection of the beam exceeds a pre-set tolerance (typically the floor height), it is automatically deleted from the model, and the static analysis is activated in order to simulate the behaviour of the remaining structure as temperatures change further.

2.4 Data transfer between static and dynamic analyses

This collapse modelling procedure involves transfer of data between the static and dynamic analyses. Modelling of a structure in fire generally starts with static analysis and switches to dynamic analysis when any instability occurs. Essential data needs to be stored and transferred between these two analytical procedures. This data includes the stress and strain states, displacements, temperature fields and fracture states of all connection components. When the static analysis is suspended, this data is imported into the dynamic analysis as its initial state. The loads on the structure are normally constant throughout the fire duration. When an instability (which is often transient) occurs, the temperature field in the structure is then held constant during the subsequent dynamic analysis. The switch back from dynamic to static analysis depends on the kinetic energy of the structure. If the kinetic energy becomes relatively small compared to the internal (strain) energy over a series of time steps, which indicates that the velocities of the structural degrees of freedom are becoming very small, this indicates that re-stabilization of the structure is being achieved, and the static procedure is resumed. The data exchange between the dynamic and static procedures involves updating the values of all the data previously passed in the opposite direction.

3 Illustrative example

In order to demonstrate the capacity of the procedure, a progressive collapse analysis of a two-storey two-span plane frame is now carried out. The dimensions of the frame are shown in Fig. 3. The cross-sections of the columns and beams in the frame are UC 254×254×73 and UB 305×127×48 respectively. It is assumed that the fire occurs in the right-hand bay of the lower storey, and that the column and beam shown within the dashed boxes in Fig. 3 are heated by the fire, with uniform temperature distribution across their sections.

The fire temperature is assumed to follow the ISO834 [28], or Eurocode 1 Part 1-2 [29], standard fire curve, and the beam's uniform temperature is estimated using the incremental heat transfer calculation given in Eurocode 3 Part 1-2 [30] for unprotected steel I-sections.

The temperature of the heated column is assumed to be half that of the heated beam. The uniformly distributed load applied to all beams is 40kN/m, giving them a load ratio (loading intensity in fire as a proportion of ambient-temperature load capacity) of 0.7. The inner (heated) column at ground floor level is lightly loaded, with a load ratio of 0.3, while the outer columns have a load ratio of 0.15.

The component-based connection element described previously is used to model all the beam-to-column connections in this model. The connection arrangement, shown in Fig. 4, includes five tension bolt rows, each of which has three components, representing the force-deformation responses of the endplate, a pair of parallel bolts and the column flange. Its two outer “bolt rows”, which work only in compression, represent the contact characteristics of the potential compression zones at the levels of the top and bottom beam flanges. Each of these rows only carries force when its deformation is compressive.

The intention of this analysis is to illustrate the capability of the developed procedure, rather than to be a study of the behaviour of a realistic structure during progressive collapse. The assumed ambient-temperature properties of the different components (endplate, bolts and column flange) in each tension row of the connection, adopted in this model, are shown in Fig. 5. The reductions of stiffness and strength of the endplate and column flange components with temperature are assumed to be those given for steel in Eurocode 3 Part 1-2. The reduction of tensile strength of the bolts with temperature is defined in line with Annex D of Eurocode 3 Part 1-2. It is assumed that the temperatures in connections J1 and J2 (see Fig. 3.) are uniform and the same as that of the heated beam.

Fig. 6 plots the axial forces of different bolt rows of the connection J1, (a) against its temperature and (b) against the rotation at left-hand beam end. The fracture of the first bolt-row triggers a force redistribution which leads to subsequent fractures of other bolt-rows, from the top to the bottom of the connection. It can be seen from Fig. 6(b) that the sequence of failures within the connection is tracked until it completely detaches. In fact the analysis carries on beyond connection fracture, until final structural collapse occurs due to column buckling at a higher temperature.

Fig. 7 shows the vertical displacement at the top of the heated column and the end rotation of the heated beam, plotted against the connection temperature. Fig. 8 illustrates the sequence of collapse of the frame. The deformation states at different temperatures are shown in this figure. The heated beam detaches at both ends from the connected column and falls down (Fig. 8(c)) at a temperature of 480°C. After it has fallen to ground level and been removed from the modelling, the analysis of the remaining structure carries on; the temperature of the heated column continues to increase until buckling occurs (Fig. 8(e)), which induces final collapse (Fig. 8(f)) of the frame. The whole of this process is tracked by the static-dynamic analysis, and the deformed configurations shown in Fig. 8 are all described in the output of numerical results. This clearly shows the efficacy of the procedure, not only in tracking the progressive failures of bolt rows, but also in simulating the structural behaviour up to final collapse.

4. Progressive collapse with connection failure

It has been observed in both the full-scale composite building fire tests at Cardington and the collapse of the WTC buildings that connections are vulnerable in fire, and that design of connections for robustness is important for structural integrity. In this section, progressive collapse analyses of steel frames in fire are carried out, firstly to investigate the influence of the fracture ductility of connections on the progressive collapse resistance of steel frames in fire, and secondly to further demonstrate the capability of the progressive collapse analysis procedure. These studies are based on two-dimensional steel frames with different connection characteristics. Obviously two-dimensional frame analyses cannot consider the effects contributed by out-of-plane members and floor slabs, which can either enhance or detract from structural integrity. The continuing concrete slab and its reinforcement can provide considerable stiffness to connections, whereas any ductility in adjacent structure can affect the robustness of the beam-to-column connections. However, two-dimensional analyses can certainly be used to investigate the load paths in the primary frames of a structure for which composite action is not dominant, such as steel frames supporting precast concrete floor systems. Within the scope of this study, the main concern is to simulate the successive ductile fractures of the components within the steel connections, and to understand the impact of the ductility of connections on structural robustness in fire. The influence of slabs and third-direction members are conservatively neglected, and these effects will have to be considered in further studies.

Fig. 9 shows the generic frames and the connections involved in this study. Frame 1 has equal-width (7.5m) bays whilst Frame 2 has a wider (10m) right-hand bay. All columns have a height of 3.6m. UC 254×254×107 and UB 356×171×51 sections are adopted for the beams and columns, respectively, in both frames. In Frame 1 the total line load on each beam at the Fire Limit State is 27kN/m, giving a load ratio of 0.7 on the beams. In Frame 2 the loads are applied to all beams to give the same load ratio as in Frame 1. This is because Frame 2, as a reference case, aims to investigate how the span of a beam affects the ductility demand on its connections. Grade 43 steel, with yield strength 275 N/mm² and Young's modulus of 210 kN/mm² at ambient temperature, is used for both beams and columns.

The axial restraint to the primary beams from continuing structure are represented by identical linear elastic springs at the left and right edges, as shown in Fig. 9. These springs have been given the same stiffness for both their tensile and compressive responses. This is clearly a simplification of the ways in which restraint to horizontal movement is generated, but gives a reasonable analogy to the restraint to the horizontal movements of the ends of a heated beam when a fire affects a single storey and the effects on its connections are the focus of attention. Depending on the real continuity conditions, the stiffness of these springs is mainly influenced either by the aggregate lateral (sway) stiffness of the upper and lower columns in adjacent bays or by the aggregate axial flexibilities of the adjacent beam and steel connections on the side of the supporting columns beyond the heated beam. As previously stated, composite action between steel beams and concrete slabs is not considered in this study. The rotational flexibility of the boundary spring is ignored, and thus full rotational restraint is assumed. The axial stiffnesses of the restraint springs are calculated as:

$$K_L = \sum_{i=1}^{\substack{number\ of \\ adjacent\ columns}} K_{l,i} \tag{1}$$

where, $K_{l,i} = \frac{12EI_{i,C}}{L_C^3}$ is the sum of the lateral stiffnesses of the upper and lower i^{th} columns in the adjacent bays either to the right or left of the column which is engulfed by the fire; $I_{i,C}$ is its sectional second moment of area and L_C is the height of a column. The sub-frames shown in Fig. 9 are imagined to be located in the central zone of a generic steel frame with 12 bays, and thus there are five bays on each side of the sub-frames. An elastic stiffness of 47.3 kN/mm, calculated from Eqn. (1), is assigned to each of the lateral springs to represent the lateral restraint provided by the columns in the side bays.

It is assumed that the members in the red-dashed boxes in Fig. 9 are exposed to the ISO843 standard fire curve, and that their temperatures are predicted according to the Eurocode 1: Part 1-2 heat transfer analysis. The gas temperature curve and the temperatures in typical members are shown in Fig. 10**.** The heated column is protected, and experiences a uniform temperature distribution across its cross-section. The temperature of the top flange of the heated beam is 0.7 times that of its web and bottom flange. Each heated connection is assumed to have the same temperature as the top flange of the heated beam to which it is connected.

Connections with different ductilities are tested in this study. The variation of ductility of the connections is achieved by providing different deformation limits to their components. Each connection contains three bolt rows, each of which is assembled from three components. Fig. 11 illustrates the properties of the components in the connections used in the present study. The aim is to demonstrate the importance of the ductility within general steel connections on improving their structural robustness in fire. Therefore, the components of the connections in this study are generally named Components 1, 2 and 3. Each of these can represent a different part for different types of connection. For instance, they would be the endplate, bolts and column flange for an endplate connection, or the endplate, bolts and reverse channel for a reverse-channel connection. The ductile deformation capacity of each component is labeled as Dt1, Dt2 and Dt3, respectively. The behaviour of the connection in compression is represented by two compression springs, each of which is assigned a simplified but representative force-displacement relationship, as shown in Fig. 11. An elastic-plastic behaviour is assumed for the compressive spring before the connection makes contact with the column flange, and it becomes completely rigid after contact with the column flange is made. Dc denotes the plastic compressive deformation capacity of the compression springs. In this study it is also assumed that the amount of plastic compressive movement is equal to the largest plastic tensile movement (Component 3) in each case. This would be approximately correct for reverse-channel and endplate connections. Five types of connection are involved in this study, although without specifying their design details. The properties of these connections are listed in Table 1.

Connection Type 1 serves as the basic type in this study, with the intention of representing typical ductility for a flush endplate connection. Connection Type 2 is more ductile than Type 1. It has been observed in experiments [18] that reverse-channel connections provided at least three times more ductility than equivalent flush endplate connections tested at the same temperature. Therefore, Type 3 and Type 4 aim to represent the connections employing a reverse channel, and Type 5 is an extreme case in which the connection has huge ductility.

3.1 Progressive failure within the connection

The complexity of the possible force distributions in a connection during a fire cannot easily be considered in normal design. The large resultant catenary force in the later stages of a fire can fail components within the connection and potentially trigger progressive fracture of all bolt rows. However, the preserved ductility within the connection can effectively reduce the catenary tensile force in the connection and, as a consequence, enhance the structural integrity under fire conditions. This effect is presented and discussed in this section, on the basis of the results of the collapse analysis for the example steel frames.

Figures 12-17 show the axial forces of the different bolt-rows of the connection CJ1 of Frame 1 with various connection types. The temperatures shown in these figures are those of the bottom flanges of the heated beams.

Behaviour in Compression:

Net compression force in the connection under fire conditions is generally induced by restrained thermal expansion. As indicated by the axial force curves in Fig. 12, thermal expansion dominates the behaviour of a connection in the early stage of a fire. With limited movement capacity in compression, the connection is pushed towards, and finally contacts, the column flange to which it is connected. The rapid growth in compression force after Point A in Fig. 12 reflects the effect of the compressive ductility in the connections on the behavior of the restrained beam; the compression force increases significantly when the compressive movement of the connection exceeds its compressive ductility. This increase of compression force may potentially buckle the restrained, beam or accumulate push-out effects of an edge column.

A comparison of the curves in Figs. 12 and 13 indicates that an increase in the compressive ductility in a connection can avert contact between the beam-end and the column flange, and consequently reduce the compressive force transmitted. It is beneficial to provide more compressive ductility to the connection in fire conditions, since this can reduce the probability of mechanisms associated with the compression force, such as local buckling in the connection vicinity, buckling of the restrained beams and push-out of the edge columns.

Behaviour in Tension:

A restrained beam under distributed lateral load is normally subjected to combinations of bending moment and shear force at ambient temperature. However, with the deflection increasing as the beam temperature rises, catenary tension starts to resist the external loads on the beam, instead of bending and shear. Tensile force exerted on the connections from the beam-ends then dominates their failure mechanisms, especially when their temperatures are very high. It can be seen from Figs. 12-16 that the axial forces in all the connections turn into tension at the peak of the fire. Table 2 lists the temperatures at which the bolt-rows initially fracture, and the failure (detachment) temperature for each type of connection.

As shown in Fig. 12 and Table 2, the initial fracture in connection Type 1 happens in the first bolt row at 650°C, after which the forces redistribute and the tensile force in the second bolt row increases. Although partial fracture has already occurred in the connection, the heated connection maintains its structural integrity, since the intact bolt-rows take the load shed by the fractured one. The fracture of the second bolt row occurs at 670°C, and the forces are again redistributed., The beam finally disengages from the column when the third bolt row breaks at about 678°C.

Table 2 also indicates that the failure temperature of the connection increases with increasing tensile ductility. Connection Type 3, which has three times the ductility of Connection Type 1, can retain its integrity until 710°C, whereas Connection Type 5, with five times the ductility of Type 1, can avoid any bolt-row fractures before the column engulfed by fire buckles. The results of the analyses clearly show that the provision of ductility in a connection enhances its robustness in fire. Another observation is that as the tensile ductility of the connection increases the difference between the initial fracture temperature and the final failure temperature of the connection reduces. This is because the higher initial fracture temperature, caused by the increased tensile ductility, degrades the intact bolt rows and hence reduces the capacity for load redistribution after the initial fracture has occurred.

Comparing the axial forces in the beams of Frames 1 and 2 reveals the effect of the beam span on the structural behaviour and ductility demand on the steel connections. The beam in Bay 1 of Frame 1 has a span of 8m, whereas the beam in Bay 1 of Frame 2 is 10m. Both of the beams are loaded to the same load ratio. Fig. 17 shows the axial forces of the bolt rows of connection CJ1 of Frame 2 with connection Type 2. The axial compression force of CJ1 in Frame 2 is larger than that of CJ1 in Frame 1 (see Fig. 13). The rapid increase (after point B) in the compressive axial force curve in Fig. 17 is, as explained previously, due to the movement in compression of the connection exceeding its movement capacity. The connection contacts the column flange, and hence the compression force in the connection rises, as a result of the restrained thermal expansion. This illustrates that higher movement capacity (ductility) in compression is needed for longer beams to avoid the large compression force in the connection due to restrained thermal expansion.

3.2 Structural behavior after fracture of a connection

Another objective of this study is to investigate the collapse sequence of the structure after a local failure, especially of connections, has been initiated. Figs. 18 and 19 show the collapse sequences of Frames 1 and 2 with connection Type 2 in fire.

In Frame 1, failure of the heated connections at both ends of the heated beams occurs almost simultaneously in Bays 1 and 2; the beams disengage from their supporting columns at a temperature of 698°C and then fall until they reach ground level (Fig. 18(c)). These beams are removed from the analysis at this point, whilst the rest of structure continues to be heated. The heated column buckles (Fig. 18(d)) at an equivalent beam bottom flange temperature of 879°C, which corresponds to a column temperature of 681°C. The failure of connections then spreads to the upper floor levels; the sequence of connection failures is shown in Fig. 18(e).

Catenary forces in the remaining beams are gradually activated as the deflection increases, and the tensile catenary forces fracture the connections in the upper floor levels. The failures progressively spread to different floors, and eventually all beams fall to ground level (Fig. 18(f)).

In Frame 2, the detachment of the central heated connection firstly happens in Bay 1 (Fig. 19(b)) and the right-hand end of the heated beam starts to fall at a temperature of 678°C. After the disengaging beam has reached ground level, the temperatures in the rest of the heated members continue to rise. Detachment of the right-hand connection occurs in Bay 2 (Fig. 19(d)) at 705°C, and the right-hand end of the beam falls to the ground. Buckling of the heated column occurs at an equivalent beam bottom flange temperature of 883°C (corresponding to 682°C column temperature). The failure sequence of the cold connections in the upper levels is shown in Fig. 19(g). It can be seen that the failure sequence of the connections, and the collapse of the frame, are effectively simulated by the progressive collapse procedure. The member positions and shapes shown in Figs. 18 and 19 are taken directly from the output data of the analysis.

5 Discussion and Conclusion

The progressive collapse analysis procedure developed in this study is capable of tracking the sequence of failures within connections and within the structure as a whole. The procedure combines alternate static and dynamic solution processes with a component-based connection element. It allows not only assessment of the robustness of individual connections in fire, taking account of the failure sequence within the connections, but also simulation of the subsequent collapse sequence of the structure. Examples of the collapse of steel frames under localized fire have been studied. It has been shown that ductility in the connections can improve the performance of the steel beams, potentially preventing the initiation of a progressive collapse in fire. Ductility of the connection's components 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 connections. Frames with different structural configurations will experience different collapse mechanisms in fire, and will require different levels of ductility in their connections to maintain their structural integrity in fire. It is clear, in general terms, that beams with longer spans require higher ductility in their connections to achieve specific levels of fire resistance; on the other hand, longer beams also generate higher compression forces in their connections in the early stages of a fire.

As a preliminary investigation of the influence of connection ductility on robustness in fire, this study has several limitations, and some points need to be highlighted for further work. The connections adopted in the analyses are generic, and investigations of the behaviour of realistic connection types and frame dimensions in fire are necessary in future studies. Both the connection model and the example frames used in this study have been two-dimensional, and are not capable of coping with out-of-plane member interactions, especially where composite floor slabs are involved. A three-dimensional approach to connection modelling is required for structural robustness assessment in fire. The impact of debris, which may generate significant dynamic effects and greater localized loading, triggering further failures within the structure, has been neglected in the analysis procedure.

The progressive collapse analysis procedure presented extends the capacity of the conventional structural fire engineering analysis tool for performance-based structural fire engineering design to the stage of tracking progressive failure sequences. This kind of analysis will allow engineers to assess the risk of progressive collapse of buildings under various fire scenarios, such as localized, spreading and travelling fires.

The estimation of ductility demand within steel connections to ensure fire resistance requirements is an importance issue if performance-based design is to prevent progressive collapse in fire conditions. With such an investigative procedure, designers can amend connection details so that their ductility is optimized to retain their integrity in fire scenarios. There may also be scope for the development of simplified methods for more traditional fire resistance design purposes.

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Figure Captions

Fig. 1. Flowchart of static-dynamic procedure when modelling failure of connections.

Fig. 2. Layout of component-based model [12] for an endplate connection.

Fig. 3. Example frame with endplate connections.

Fig. 4. Endplate connection at J1 and J2 (see Fig. 3), and its component-based model.

Fig. 5. Force-deformation curves for components of "tension" rows at ambient temperature.

Fig. 6. Axial forces in different components of connection J1 (a) against temperature and (b) against rotation at left-hand beam end.

Fig. 7. Beam end rotation at J1 and displacement at the top of column C1 against temperature.

Fig. 8. Progressive failure of the example frame.

Fig. 9. Two-dimensional frames and connection component model.

Fig. 10. Temperatures in the columns and the beams

Fig. 11. Illustration of the connections' properties: (1) in tension; (2) in compression.

Fig. 12. Axial forces in the components in connection of Type 1 (Frame 1)

Fig. 13. Axial forces in the components in connection of Type 2 (Frame 1)

Fig. 14. Axial forces in the components in connection of Type 3 (Frame 1)

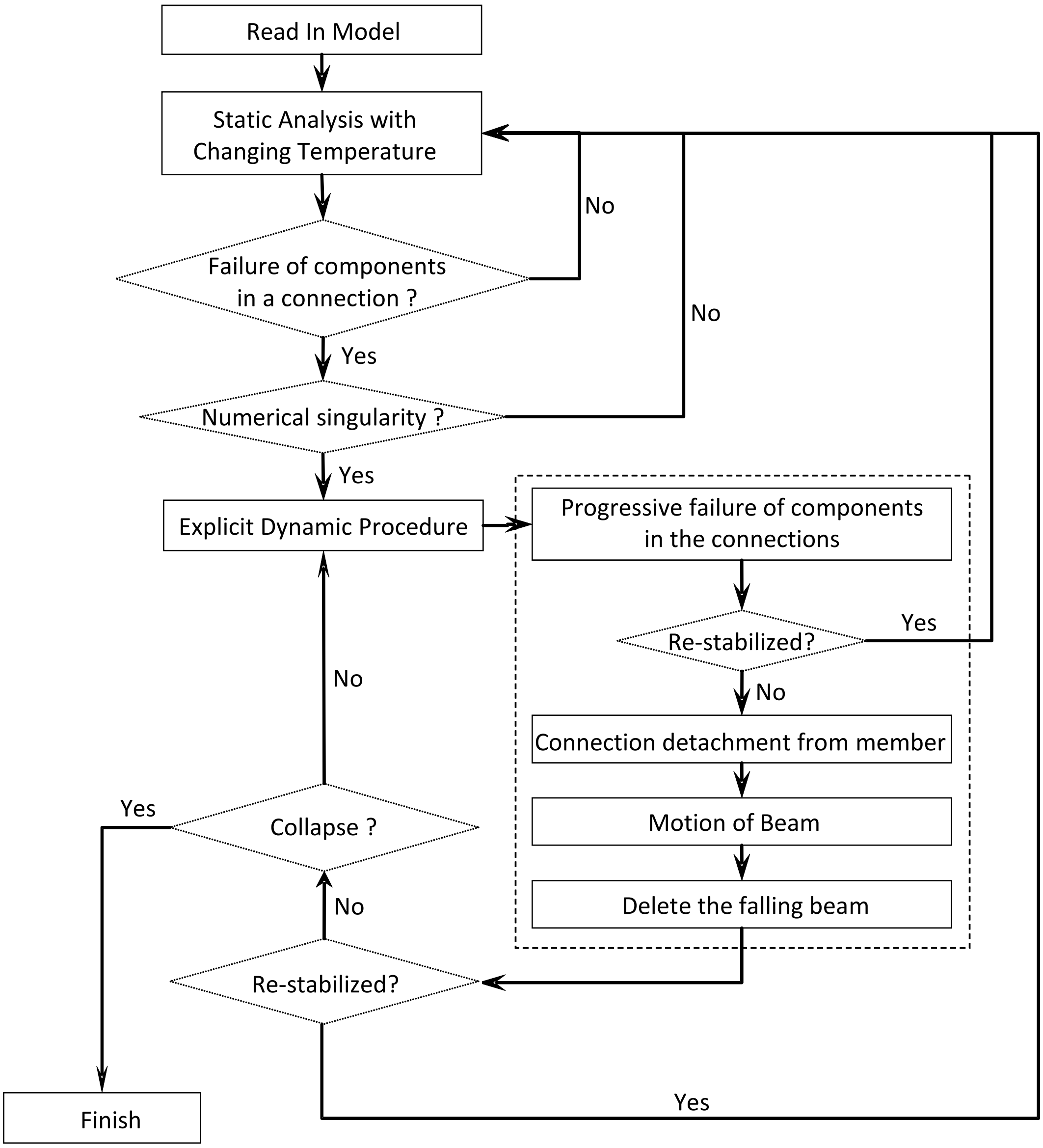
Fig. 15. Axial forces in the components in connection of Type 4 (Frame 1)

Fig. 16. Axial forces in the components in connection of Type 5 (Frame 1)

Fig. 17. Axial forces in the components in connection of Type 2 (Frame 2)

Fig. 18. Collapse sequence of Frame 1

Fig. 19. Collapse sequence of Frame 2



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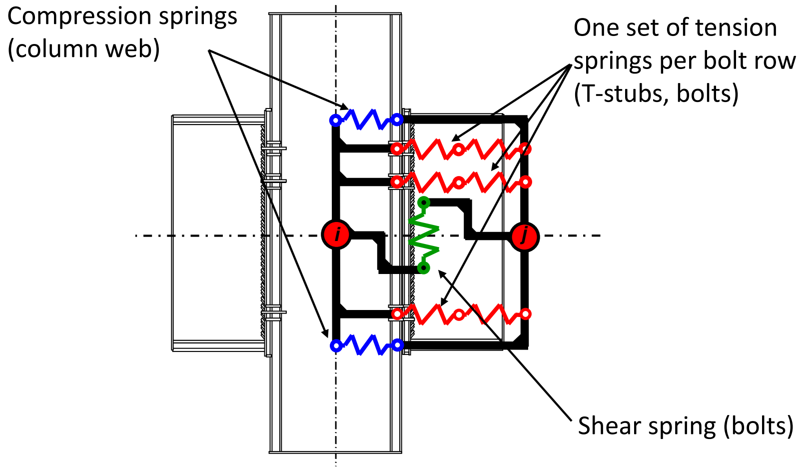


Fig. 2. Layout of component-based model [12] for an endplate connection.

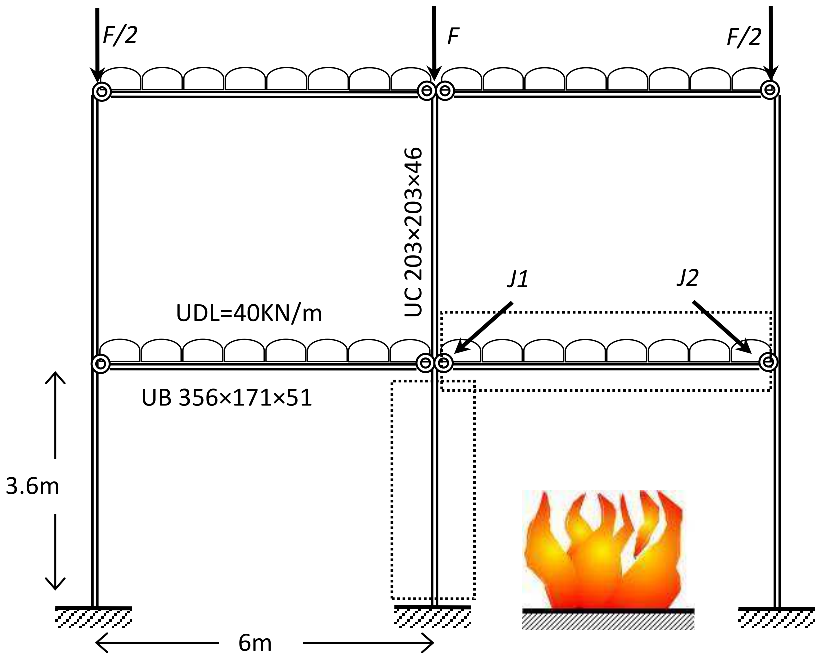


Fig. 3. Example frame with endplate connections.

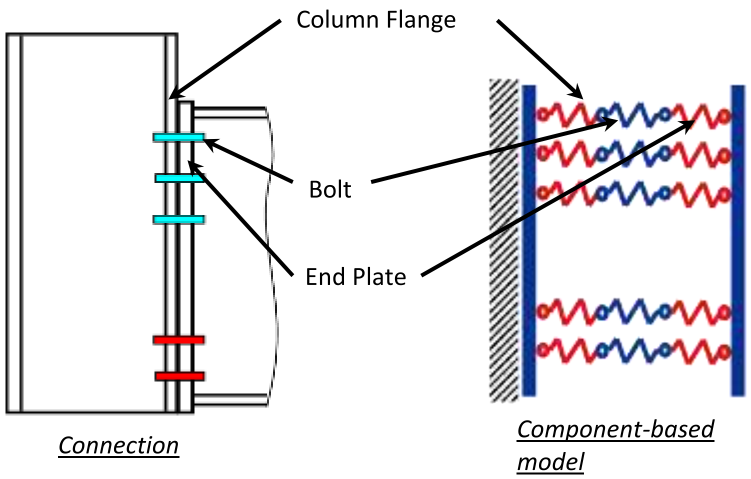
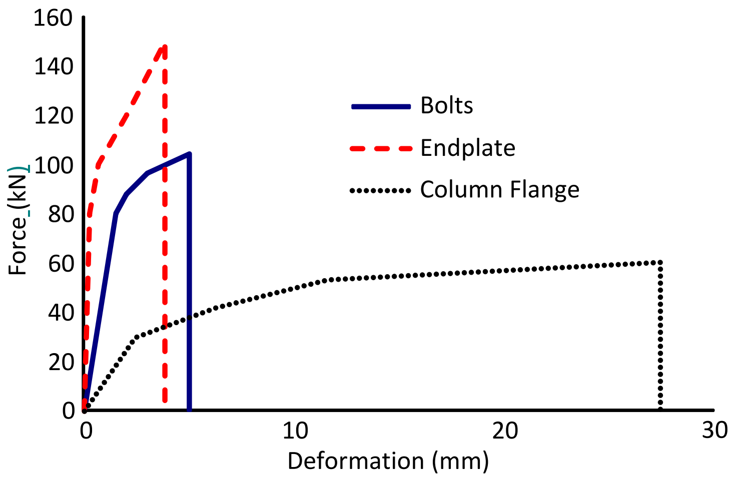


Fig. 20. Endplate connection at J1 and J2 (see Fig. 3), and its component-based model.



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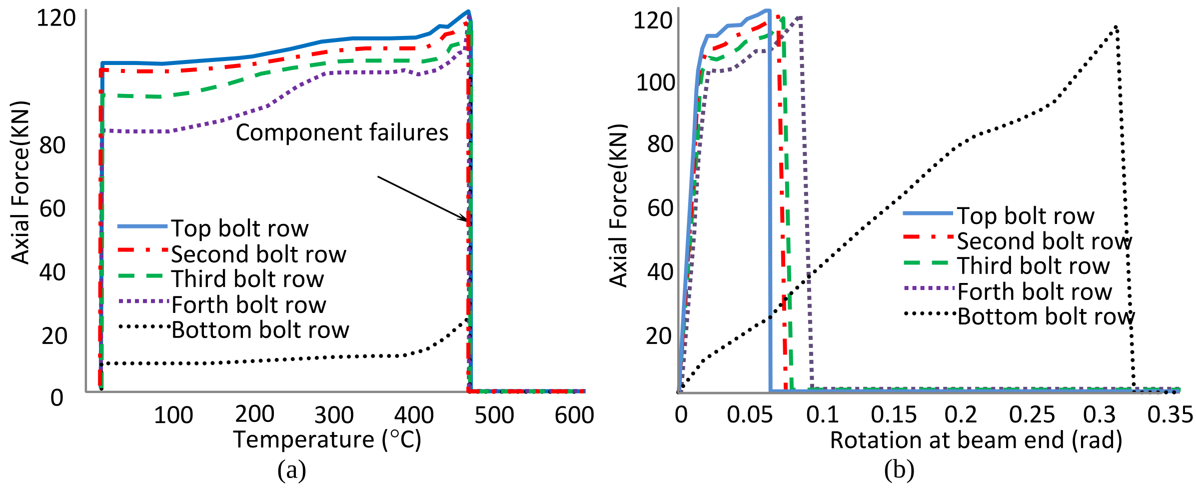

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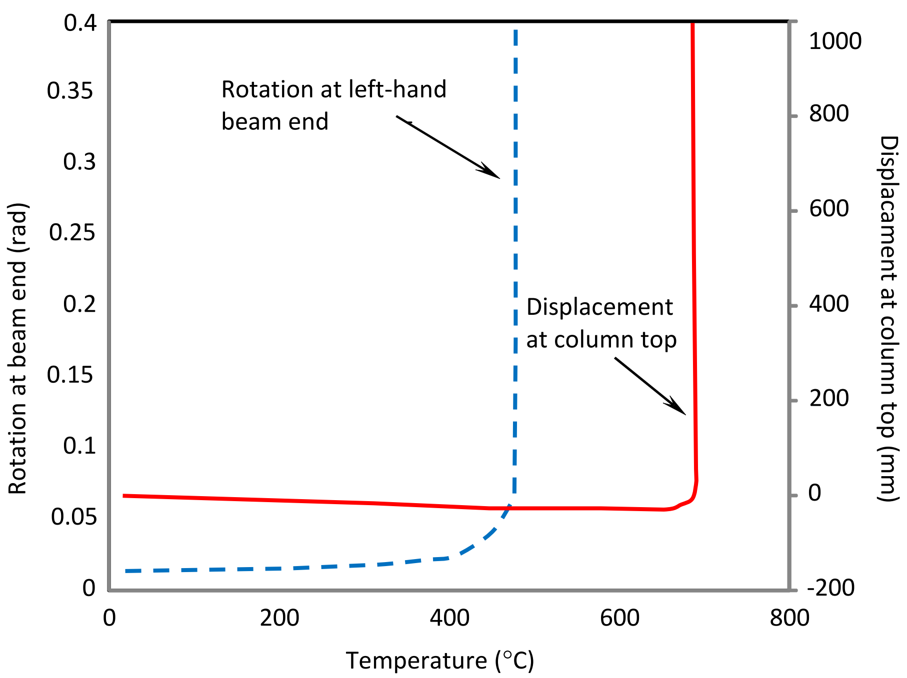
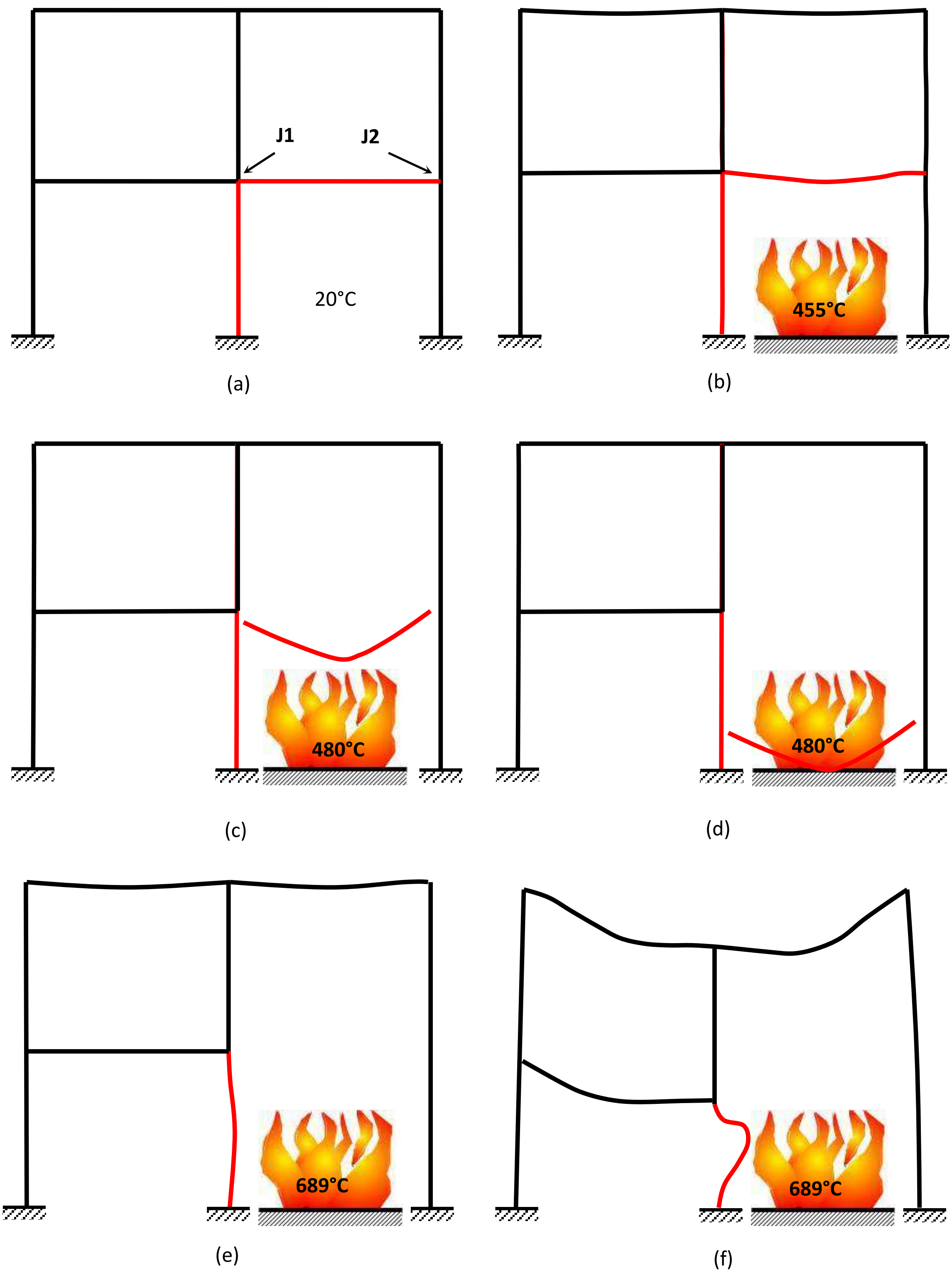


Fig. 7. Beam end rotation at J1 and displacement at the top of column C1 against temperature.



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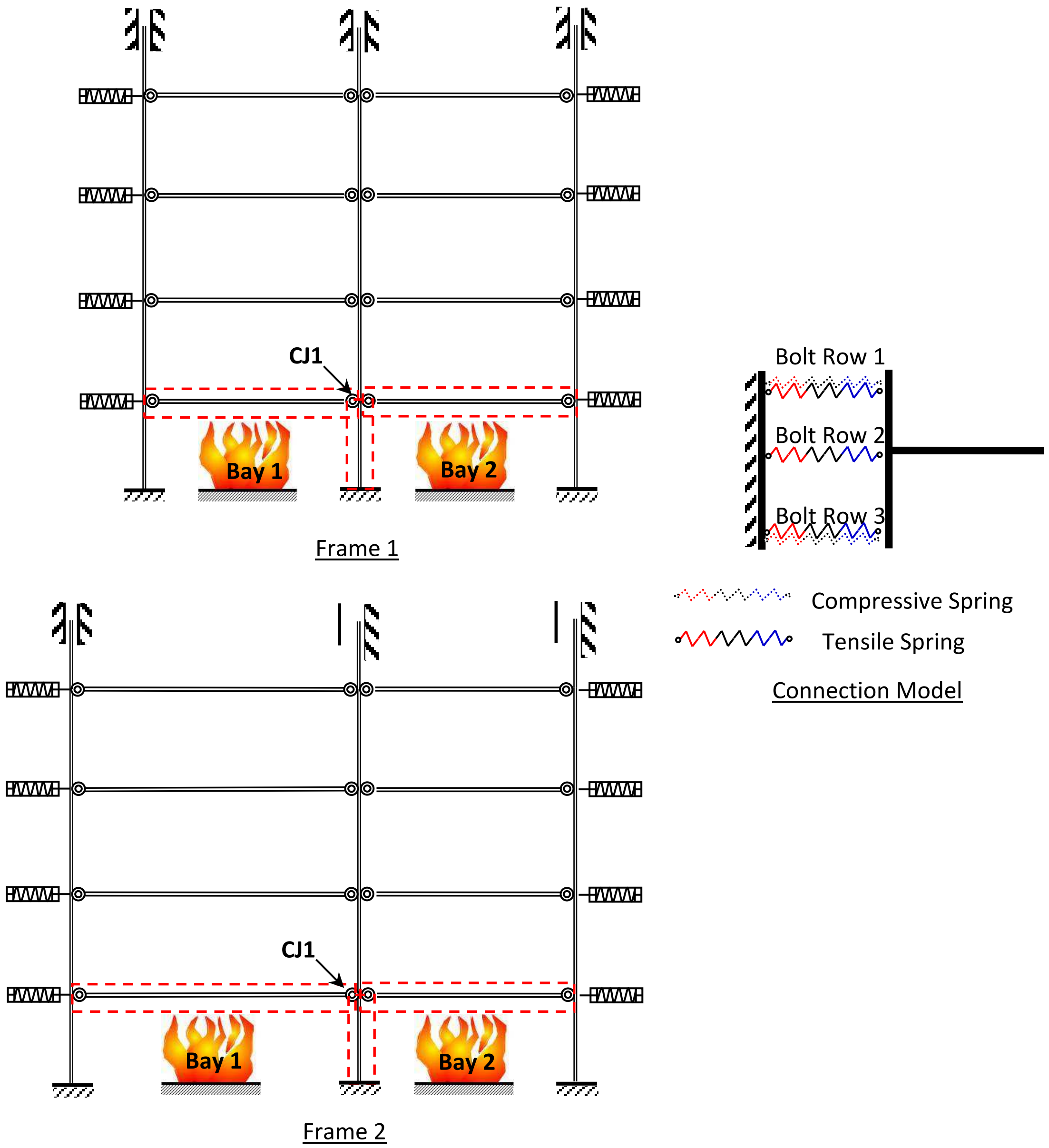


Fig. 9. Two-dimensional frames and connection component model.

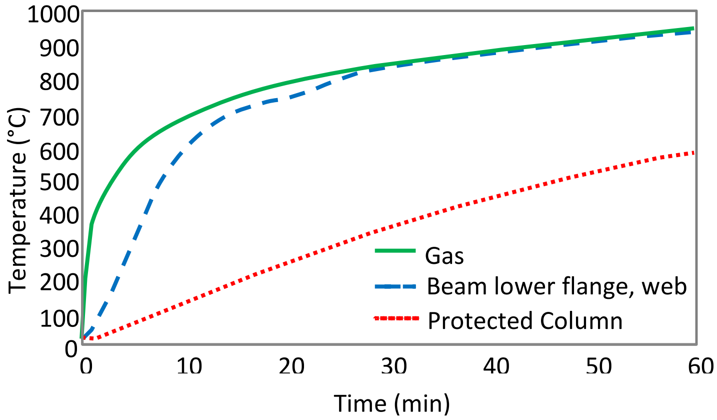
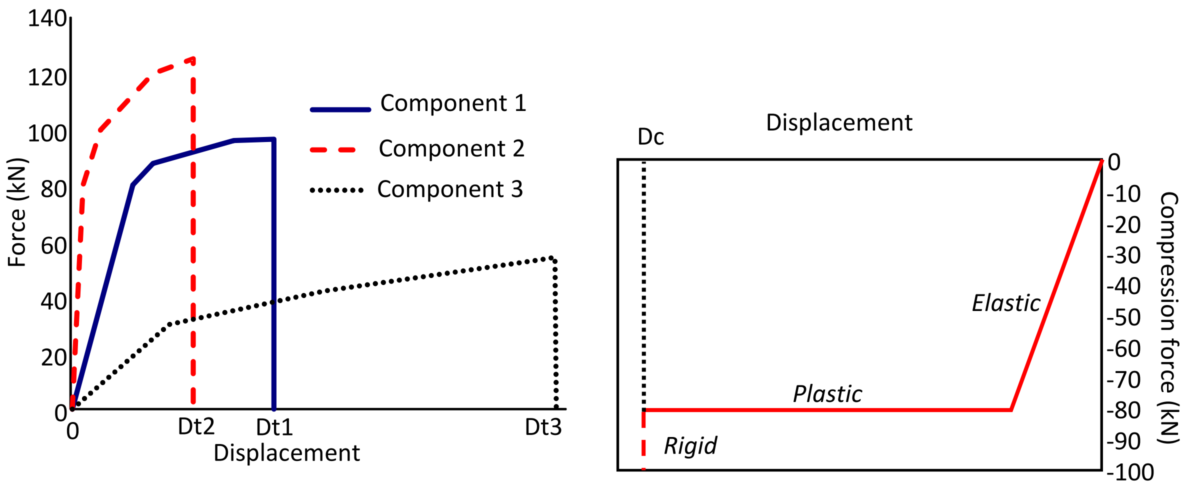


Fig. 10. Temperatures in the columns and the beams



(1) (2)

Fig. 11. Illustration of the connections' properties: (1) in tension; (2) in compression.

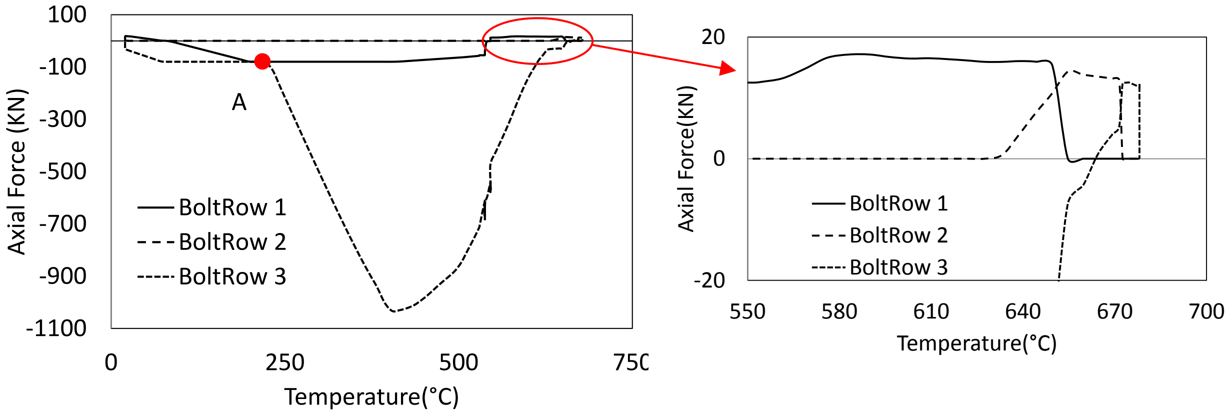


Fig. 12. Axial forces in the components in connection of Type 1 (Frame 1)

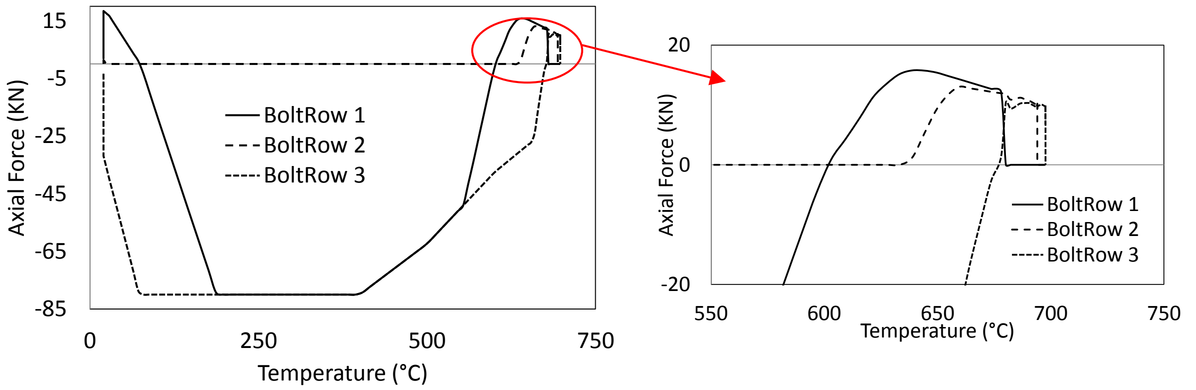


Fig. 13. Axial forces in the components in connection of Type 2 (Frame 1)

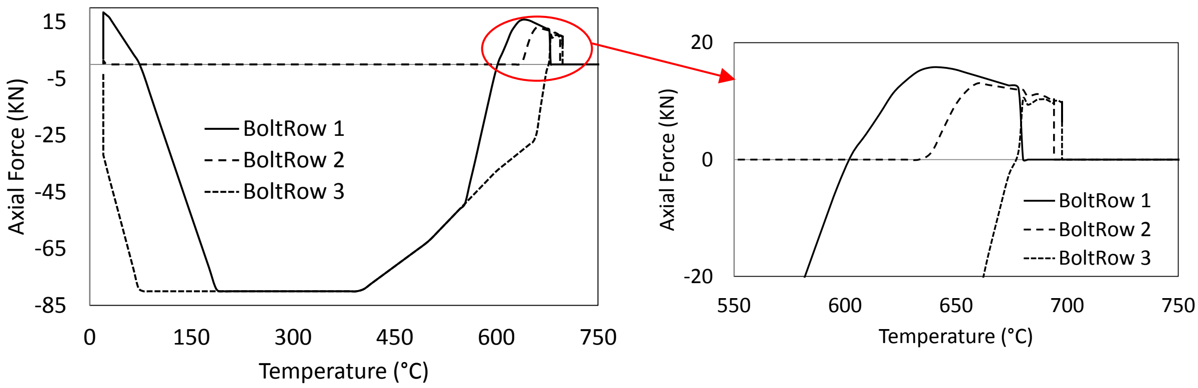


Fig. 14. Axial forces in the components in connection of Type 3 (Frame 1)

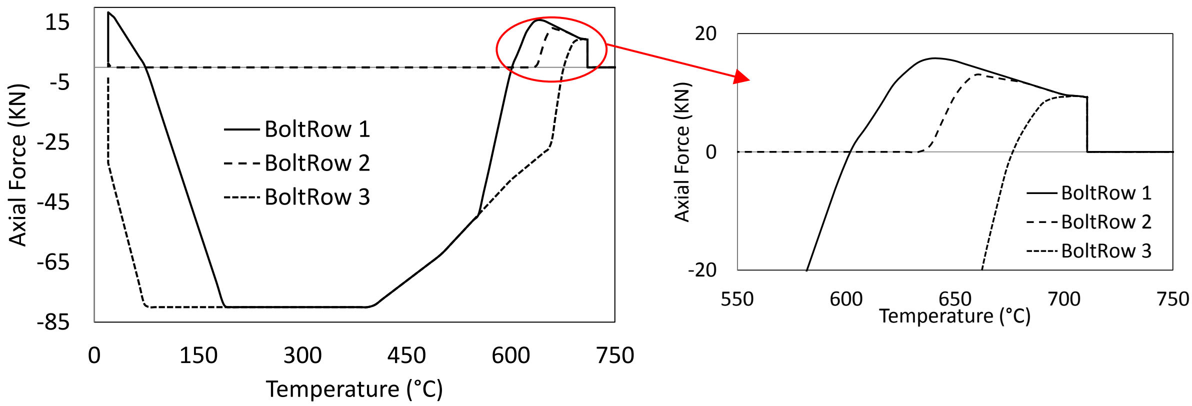


Fig. 15. Axial forces in the components in connection of Type 4 (Frame 1).

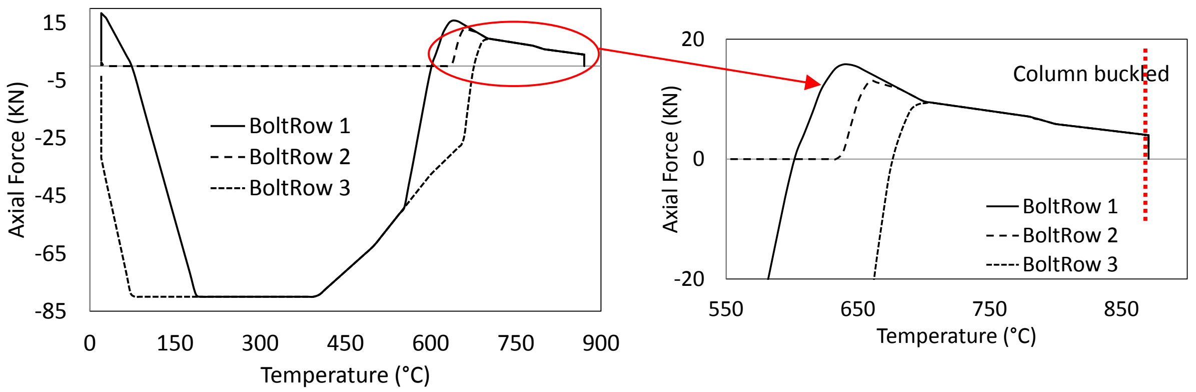


Fig. 16. Axial forces in the components in connection of Type 5 (Frame 1).

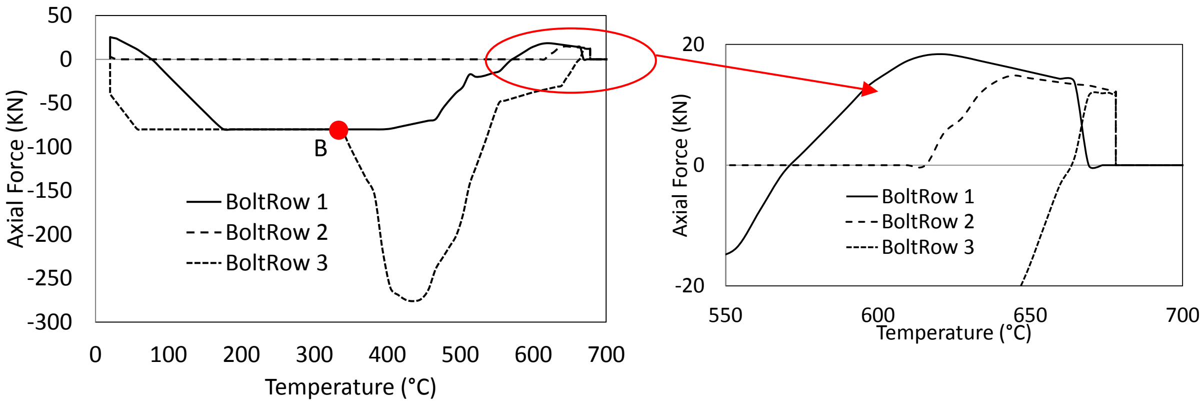
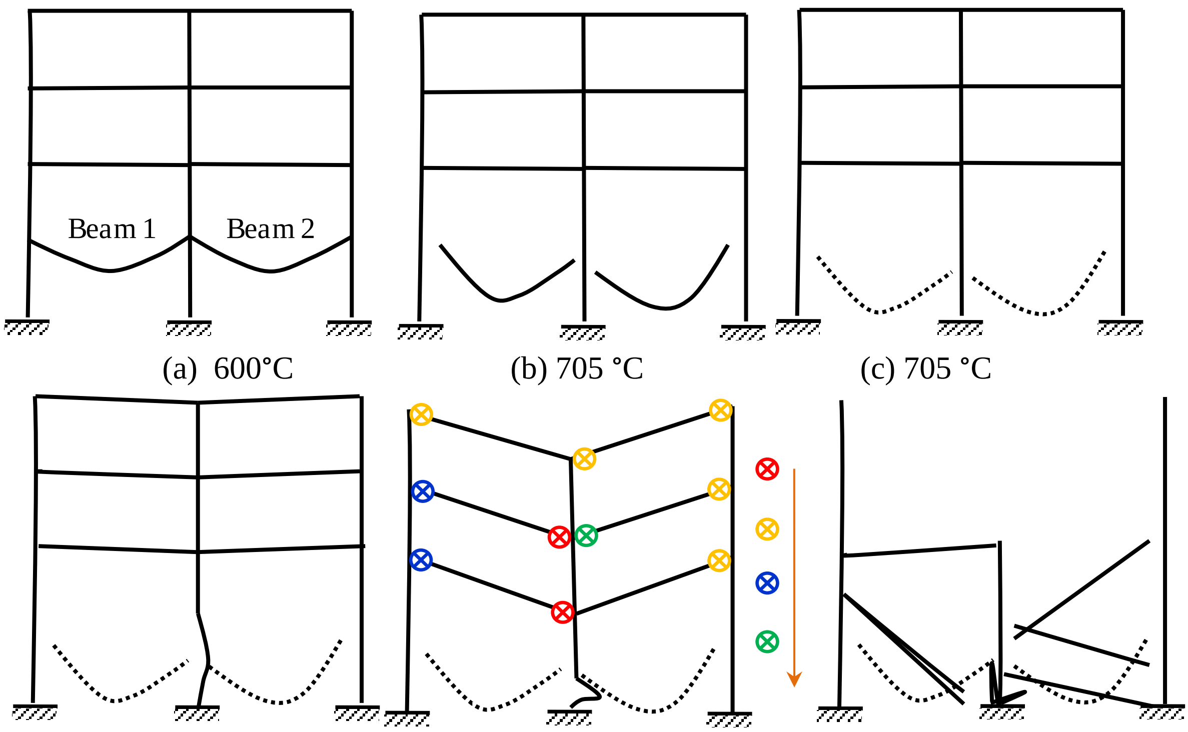
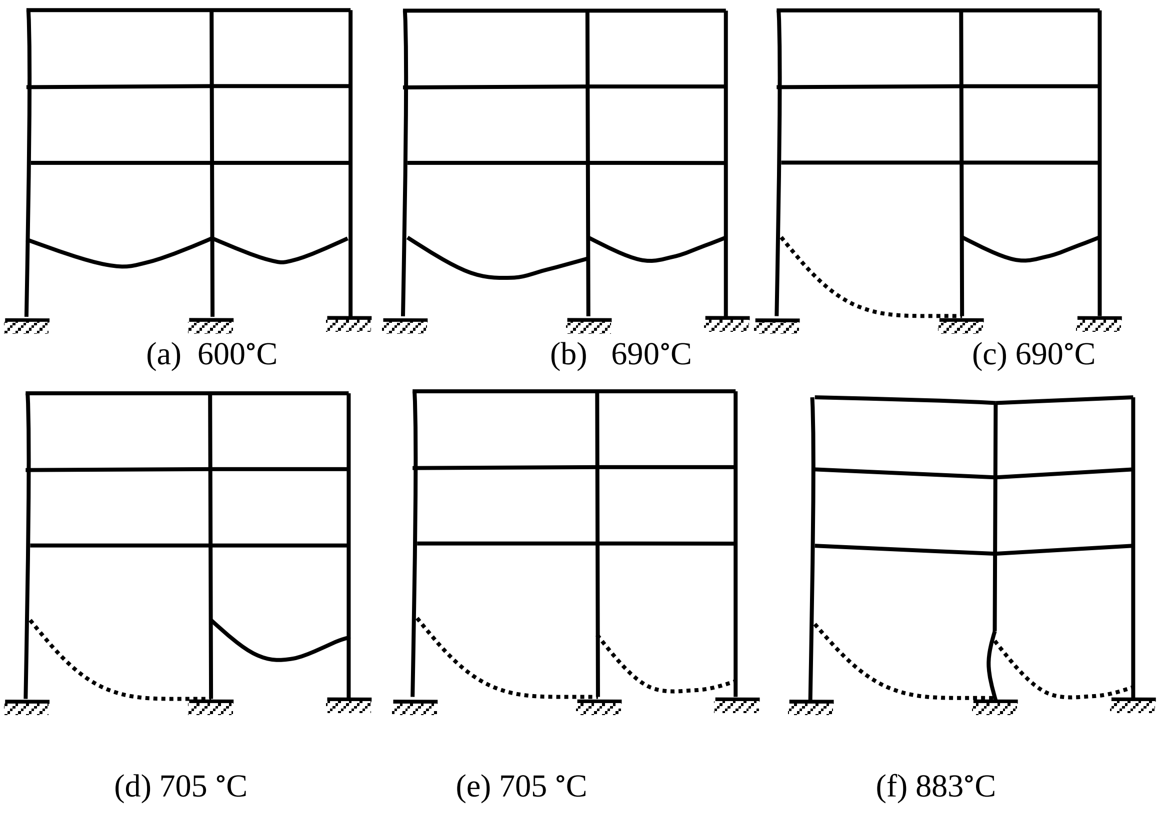


Fig. 17. Axial forces in the components in connection of Type 2 (Frame 2).



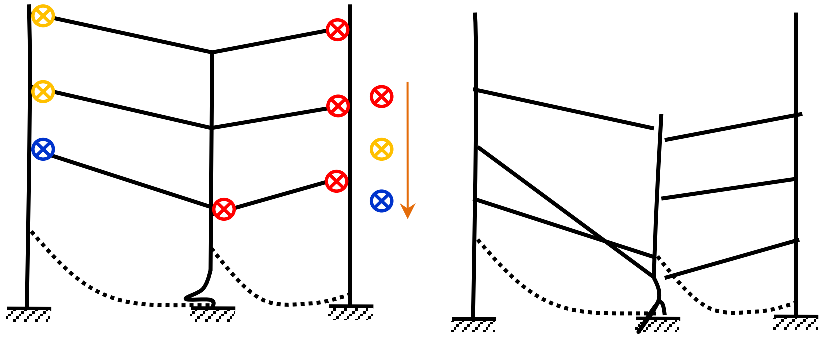
(d) 879°C (e) 879°C (f) 879°C

Fig. 18. Collapse sequence of Frame 1.



(a) 600°C (b) 690°C (c) 690°C

(d) 705 °C (e) 705 °C (f) 883°C



(g) 883°C (h) 883°C

Fig. 21. Collapse sequence of Frame 2.

Table List:

Table 1 Plastic movement capacity of components of different connection types

Connection Type	Dt1(mm)	Dt2(mm)	Dt3(mm)	Dc(mm)
Type 1	5	3	12	12
Type 2	10	6	24	24
Type 3	15	9	36	36
Type 4	20	12	48	48
Type 5	25	15	60	60

Table 2 Fracture temperatures of the bolt-rows and failure temperatures of the connections

Connection Type	Bolt Row 1 (ºC)	Bolt Row 2 (ºC)	Bolt Row 3 (ºC)	Failure Temperature (ºC)
Type 1	655	670	678	678
Type 2	680	694	698	698
Type 3	697	701	701	701
Type 4	710	710	710	710
Type 5	---	---	---	No failure before column buckles