ROBUSTNESS OF CONNECTIONS TO CONCRETE-FILLED STEEL TUBULAR COLUMNS UNDER FIRE DURING HEATING AND COOLING

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NOMENCLATURE

ACRONYMS
BFRS   - Bolts using Fire Resistant Steel
BLT    - Beam Limiting Temperature
CES    - Concrete Encased Steel
CFP    - Connection Fire Protected
CFRS   - Connection Components using Fire Resistant Steel
CFST   - Concrete-Filled Steel Tube
CHS    - Circular Hollow Section
FE     - Finite Element
FEM    - Finite Element Modeling
FR     - Fire Resistant
LVDT   - Linear Variable Differential Transformer
RHS    - Rectangular Hollow Section
SHS    - Square Hollow Section
UB     - Universal Beam
UC     - Universal Column

NOTATION
A     - Cross-sectional area
A_m  - Surface area of the element per unit length
A_p  - Area of the fire protection material per unit length of the element
A_s  - Stressed area of bolt shank
b     - Beam flange width
b_i   - Minimum dimension of the concrete core (mm)
\( c_a \)  - Specific heat of steel
\( c_p \)  - Specific heat of the fire protection material
\( c \)   - Damping factor
\( d \)   - Diameter of bolt
\( D_h \) - Diameter of hole
\( d_p \) - Thickness of the fire protection material
E     - Elastic (Young’s) modulus
\( E_{\theta} \) - Elastic (Young’s) modulus, at temperature, \( \theta \)
\( e_2 \) - End distance
\( F \) - External forces
\( F_v \) - Viscous forces
\( f_{c,\theta} \) - Concrete cube strength, at temperature \( \theta \)
\( f_{cu} \) - Concrete cube strength
\( f_p \) - Steel strength, proportional limit
\( f_{p,\theta} \) - Steel strength, proportional limit, at temperature \( \theta \)
\( f_{ub} \) - Ultimate strength of the bolts
\( f_u \) - Ultimate strength of steel plate
\( f_y \) - Yield steel strength, general
\( f_{y,a} \) - Yield steel strength, ambient temperature
\( f_{y,\theta} \) - Yield steel strength at temperature \( \theta \)
\( h \) - Beam depth
\( h_{net} \) - Design value of the net heat flux per unit area
\( I \) - Internal forces
\( k_{b,\theta} \) - Strength reduction factor for bolts
\( k_{E,\theta} \) - Reduction factors for elastic (Young’s) modulus at temperature \( \theta \)
\( k_{p,\theta} \) - Reduction factors for steel strength, proportional limit, at temperature \( \theta \)
\( k_{sh} \) - Correction factor for the shadow effect
\( k_{y,\theta} \) - Reduction factors for yield steel strength at temperature \( \theta \)
\( L \) - Beam length
\( M^* \) - Artificial mass matrix calculated with unit density
\( M_a \) - Bending moment in the column caused by catenary action force
\( M_p \) - Column bending moment resistance at ambient temperature
\( M^* \) - Artificial mass matrix calculated with unit density
\( N_a \) - Applied axial load in column
\( N_p \) - Column axial compression resistance at ambient temperature
\( P \) - Spacing between bolts
\( T \) - Temperature
\( t \) - Time
\( t_0 \) - Tubular column thickness
\( t_1 \) - End-plate thickness
\( t_2 \) - Reverse channel web thickness
\( t_{ce} \) - Increase in steel tube thickness for temperature calculation (mm)
\( t_f \) - Beam flange thickness
\( t_{\text{max}} \) - Maximum heating period
\( t_p \) - Thickness of the plate
\( t_s \) - Original steel tube thickness (mm)
\( t_w \) - Beam web thickness
\( V \) - Volume of the element per unit length

**GREEK SYMBOLS**

\( \Delta T \) - Temperature difference between the top and bottom flanges
\( \Delta t \) - Increment of time
\( \Delta \theta_{a,t} \) - Change in temperature
\( \Delta \theta_{g,t} \) - Increase in gas temperature during the time interval \( \Delta t \)
\( \alpha \) - Thermal expansion coefficient of steel
\( \gamma_{\text{Mb}} \) - Partial safety factor for the bolt
\( \varepsilon_{c1,0} \) - Concrete strain at temperature, \( \theta \)
\( \varepsilon_{\text{cu1},0} \) - Concrete ultimate strain at temperature, \( \theta \)
\( \varepsilon_{\text{nom}} \) - Nominal strain
\( \varepsilon_{\text{true}} \) - True strain
\( \varepsilon_{p,0} \) - Proportional strain limit at temperature, \( \theta \)
\( \varepsilon_{u,0} \) - Ultimate strain at temperature, \( \theta \)
\( \varepsilon_{y,0} \) - Yield strain at temperature, \( \theta \)
\( \theta_{g,t} \) - Gas temperature at time \( t \)
\( \theta_{\text{max}} \) - Gas temperature at the end of the heating phase
\( \theta_{a,t} \) - Steel temperature at time \( t \)
\( \lambda_p \) - Thermal conductivity of the fire protection system
\( \rho_a \) - Density of steel
\( \rho_p \) - Density of the fire protection material
\( \sigma_{\text{nom}} \) - Nominal stress
\( \sigma_{\text{true}} \) - True stress
\( \nu \) - Vector of nodal velocities
ABSTRACT

Joint behaviour in fire is currently one of the most important topics of research in structural fire resistance. The collapse of World Trade Center buildings and the results of the Cardington full-scale eight storey steel framed building fire tests in the UK have demonstrated that steel joints are particularly vulnerable during the heating and cooling phases of fire. The main purpose of this research is to develop robust joints to CFT columns that are capable of providing very high rotational and tying resistances to make it possible for the connected beam to fully develop catenary action during the heating phase of fire attack and to retain integrity during the cooling phase of fire attack.

This research employed the general finite element software ABAQUS to numerically model the behaviour of restrained structural subassemblies of steel beam to concrete filled tubular (CFT) columns and their joints in fire. For validation, this research compared the simulation and test results for 10 fire tests previously conducted at the University of Manchester. It was envisaged that catenary action in the connected beams at very large deflections would play an important role in ensuring robustness of steel framed structures in fire. Therefore, it was vital that the numerical simulations could accurately predict the structural behaviour at very large deflections. In particular, the transitional behaviour of the beam from compression to catenary action presented tremendous difficulties in numerical simulations due to the extremely high rate of deflection increase. This thesis will explain the methodology of a suitable simulation method, by introducing a pseudo damping factor. The comparison between the FE and the experimental results demonstrates that the 3-D finite element model is able to successfully simulate the fire tests.

The validated ABAQUS model was then applied to conduct a thorough set of numerical studies to investigate methods of improving the survival temperatures under heating in fire of steel beams to concrete filled tubular (CFT) columns using reverse channel connection. This study investigated five different joint types of reverse channel connection: extended endplate, flush endplate, flexible endplate, hybrid flush/flexible endplate and hybrid extended/flexible endplate. The connection details investigated include reverse channel web thickness, bolt diameter and grade, using fire-resistant (FR) steel for different joint components (reverse channel, end plate and bolts) and joint temperature control. The effects of changing the applied beam and column loads were also considered. It is concluded that by adopting some of the joint details to improve the joint tensile strength and deformation capacity, it is possible for the beams to develop substantial catenary action to survive very high temperatures. This thesis also explains the implications on fire resistant design of the connected columns in order to resist the additional catenary force in the beam.

The validated numerical model was also used to perform extensive parametric studies on steel framed structures using concrete filled tubular (CFT) columns with flexible reverse channel connection and fin plate connection to find means of reducing the risk of structural failure during cooling. The results lead to the suggestion that in order to avoid connection fracture during cooling, the most effective and simplest method would be to reduce the limiting temperature of the connected beam by less than 50°C from the limiting temperature calculated without considering any axial force in the beam.
DECLARATION

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DEDICATION

For my parents

My loving mother - Zineb Hanafi

My dear father - Ahmed Elkarim Elsawaf
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Sherif Elsawaf

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LIST OF PUBLICATIONS


CHAPTER 1 – RESEARCH BACKGROUND

1.1 RESEARCH BACKGROUND

When a steel structure is subjected to fire, its load-carrying capacity degrades. One of the main reasons of this degradation is the reduction in both stiffness and strength of the material. Another one is interactions in the structure when the structure has restraints. These degradations can cause structural failure in fire. Whilst it is now feasible to include the effects of material degradation in quantification of structural fire resistance of isolated structural elements, fire induced progressive collapse, involving structural interactions poses great concern. A critical issue is that of joint behaviour in fire and interactions between joints and other structural elements.

Figure 1.1 (Wang 2002) shows qualitatively the behaviour of a restrained beam at elevated temperatures. As can be seen in this figure, a longitudinally restrained beam under fire acts totally differently from a beam without longitudinal restraint. It passes through three main stages. At the initial stage of fire the steel beam starts to experience compressive internal forces due to the restraint to thermal expansion and as the length of the beam increases. With the increase of the beam temperature, the steel begins to lose its strength and the internal axial force transfers from compressive into tensile, and then the connections begin to support the steel beam by resisting pull-in forces as well as the vertical shear force. The tensile force in the beam is generated when the beam’s shortening under very large vertical deflection overtakes its thermal expansion. Under this condition, the connected beam will exert forces on the joints that would not have been designed for in conventional design. If the joints have sufficient strength and ductility, it is possible for the connected beam to develop catenary action in fire to achieve very high fire resistance. On the other hand, the presence of a tensile axial load in the connected beam can cause the joints to fracture, increasing the risk of progressive collapse. The connections play the most important role to retain structural integrity of the whole assembly of structural members. It is vital that methods are developed to enable the connections to perform in the desired way, i.e. possessing tensile resistance and deformation capacity.
During the well publicised Cardington structural fire research programme, parts of some connections were observed to have suffered fracture during cooling, as shown in Figure 1.2. This was caused by tensile force developed in the connected beams when their contraction under cooling was restrained by the structure. This led some to believe that fire spread and structural collapse may occur during cooling and therefore this issue should be considered in structural fire engineering design. It is important to investigate the risk of connection fracture during cooling and methods of reducing such risk.

Figure 1.1: Illustrative behaviour of a rotationally and axially restrained beam in fire (Wang 2002)

Figure 1.2: Failure of flexible endplate and fin plate joints after Cardington fire test (Wald et al. 2006)
1.2 ORIGINALLITY OF RESEARCH

The collapse of World Trade Center buildings and the results of the Cardington full-scale eight storey steel framed building fire tests in the UK have demonstrated that steel joints are particularly vulnerable during the heating and cooling phases of fire. Joint behaviour is the key issue to be resolved and is the focus of this research.

The majority of research work that has been carried out to study the behaviour of beam-column joints in fire was focused on testing and modelling isolated joint configurations, where the effect of structural continuity and the presence of axial forces are usually ignored. Therefore, joints were mainly assumed to act in bending and the major research efforts were directed at establishing moment–rotation responses in the absence of axial force in the beam. Also, attention was focused on simulating joint response before very large deformations, so any inaccuracy in predicting joint behaviour during the very large deformation stage would often be overlooked. In contrast, in the fire tests conducted at the University of Manchester, the joint forces were variable throughout the fire exposure, as would be expected in realistic structures. This requires that the large deflection phase of structural behaviour be accurately modelled.

The recent publication by Dai et al (2010) appears to be the only one to have included the detailed behaviour of realistic steel beam/column joints in structures in which the connected beams experienced large deformations and the joints had variable forces in fire. The study was based on common beam/column connections using open sections.

The current research will focus on concrete filled tubular (CFT) columns, which have a number of advantages including attractive appearance, high structural load carrying capacity and high fire resistance. However, use of CFT columns has to overcome the problem of making connections which are more difficult with tubular sections. A recent experimental study by Ding and Wang (2007) suggests that the so-called reverse channel connection, in which the two legs of a channel are welded to the tubular column face and the web connected to conventional flexible/flush/extended endplate on the beam side, can develop substantial fire resistance, yet remains moderate in fabrication cost. Figure 3.2 shows a sketch of the reverse channel connection.
The tests of Ding and Wang (2007) will be used as the basis of validation of the simulation method to be developed in this research. In addition to understanding structural behavior in fire, it is important to investigate practical means of achieving a robust structure. There has not been much research on this and this will be the main aim of the current research project.

Figure 1.3: Typical reverse channel connection using a flexible endplate

1.3 OBJECTIVES AND METHODOLOGY OF RESEARCH

This study has the following objectives:

1) To develop and validate a three-dimensional (3-D) FE model using ABAQUS software for modelling the behaviour of restrained structural subassemblies of steel beam to concrete filled tubular (CFT) columns and their joints under fire conditions, including both the heating and cooling phases of fire.

2) To investigate methods of enhancing the strength and deformational capacities of reverse channel connections to CFT columns to improve structural fire resistance, particularly to enhance the ability of the connected beam to develop substantial catenary action at high temperatures during the heating phase of fire attack.
3) To conduct extensive numerical simulations to explore different methods of enabling reverse channel connections and fin plate connections to survive the entire fire exposure, particularly during the cooling phase of fire attack.

This research will focus on steel framed structures using concrete filled tubular columns (CFT). Recently, a series of 10 fire tests were carried out on steel beam to CFT column assemblies at the University of Manchester by Ding and Wang (2007). This research will start from this experimental research and carry out numerical simulations. The numerical simulations will first validate the numerical models and then the validated numerical models will be used to perform extensive parametric studies of joint and structural assembly behaviour under fire conditions, including both the heating and cooling phases of fire.

1.4 THESIS STRUCTURE

Chapter 2 of this thesis is a literature review that gives a general introduction to concrete-filled tube (CFT) columns and their connections, followed by an introduction to their behaviour, at both ambient and elevated temperatures. Research on behaviour of joints and restrained steel beams in fire is highlighted.

Chapter 3 presents a brief description of the 10 structural fire tests conducted at the University of Manchester and presents the methodology employing the general finite element software ABAQUS to numerically model the behaviour of restrained structural subassemblies of steel beam to concrete filled tubular (CFT) columns and their joints in fire. Since this model is subsequently used as the tool for further analyses in this research, this chapter includes the results of sensitivity studies on finite element mesh, boundary conditions of the structural assembly and the effects of joint details including gap size between the bolt and the plate.

Chapter 4 presents the validation results by comparing the simulation and test results for the 10 fire tests using different types of connections recently conducted at the University of Manchester.
Chapter 5 introduces different ways of enhancing the strength and deformational capacities of reverse channel connections to CFT columns to prolong the catenary action stage in the connected beam. The following methods were investigated:

- Using hybrid connections;
- Using fire-resistant steel in joint components;
- Fire protection of joint components;
- Joint detailing.

Chapters 6 and 7 focus on the behaviour of restrained structural subassemblies of steel beam to concrete filled tubular (CFT) columns and their joints during cooling, Chapter 6 considering reverse channel connection and chapter 7 for fin plate connection.

The thesis is concluded with Chapter 8, in which the main conclusions from the research as a whole are presented, as well as recommendations for areas of future study.
CHAPTER 2 – LITERATURE REVIEW

2.1 INTRODUCTION

Based on the objectives of this research as outlined in chapter 1, the literature review will include the following aspects:

(1) An introduction to CFT column behaviour, at both ambient and elevated temperatures;
(2) Behaviour of restrained steel beams in fire;
(3) Behaviour of joints, particularly, joints to CFT columns in fire;
(4) Behaviour of structural frames in fire, including during the cooling stage, and fire induced progressive structural collapse;
(5) Numerical simulations of joints and structural assemblies in fire.

2.2 INTRODUCTION OF CONCRETE-FILLED TUBE (CFT) COLUMN BEHAVIOUR

A concrete-filled tube (CFT) column consists of a steel tube filled with concrete. The concrete core adds stiffness and compressive strength to the tubular column and reduces the potential for inward local buckling. On the other hand, the steel tube acts as longitudinal and lateral reinforcement for the concrete core helping it to resist tension, bending moment and shear and providing confinement for the concrete. The steel tube also prevents the concrete core from spalling under fire attack. Figure 2.1 shows a number of types of concrete filled cross-sections (Artiomas 2007).
CHAPTER 2 – LITERATURE REVIEW

Figure 2.1: Various types of concrete filled columns: (a) Concrete-Filled Steel Tube (CFST) - (b) combination of Concrete Encased Steel (CES) and CFST- (c) hollow CFST sections - (d) double skin sections (Artiomas 2007)

Research on CFT columns dates back to 1960s. As such, comprehensive understanding on structural behaviour of CFT columns, at both ambient and elevated temperatures, is now available and design guides are well established (i.e. BS 5400 Part 5 (BSI 1979), Eurocode 4 part 1.2 (CEN 2004) and Corus Tubes (Hicks and Newman 2002).

2.3 BEHAVIOUR OF CFT COLUMNS AT HIGH TEMPERATURE

Wang (2002) has presented a qualitative description of the performance of CFT columns under fire. Figure 2.2 shows a typical response of the recorded axial deformation-time relationship. It can be divided into four parts: (1) a phase of steady increase in the column expansion (A-B) is followed by (2) a sharp contraction (B-C) and (3) then gradual contraction (C-D) in the column axial deformation; and (4) the column experiences another sharp contraction (D-E) before failure.

This type of behaviour may be explained by considering the temperatures and resistances of the steel tube and the concrete. Initially, the steel tube and the concrete core share the applied load in composite action. During the early stages of the fire attack, because the steel is at a much higher temperature, it expands faster than the concrete. Meanwhile, the applied load is now mainly resisted by the steel tube and the first phase corresponds to thermal expansion of the steel. As the steel temperature increases, it loses its load carrying capacity and the column suddenly contracts due to
buckling of the steel tube. This is reflected in the second stage behaviour and is often accompanied by local bulging of the steel tube. If the concrete core has sufficient load carrying capacity, the applied load will be shed from the steel tube to the concrete core when the steel tube has contracted in length to the level of the concrete core. The column response is now characterized by a gradual contraction until the applied load exceeds the combined resistance of the steel tube and the concrete core at much higher temperatures. Since the load is mainly resisted by the concrete core, the thickness of the steel tube has very little influence on the fire resistance time of the composite column.

![Figure 2.2: Typical time-axial deformation response of a concrete filled column (Wang 2002)](image)

Although there are still some issues with developing an accurate method for the design of CFT columns in fire, the design of CFT columns in fire may be treated in a similar way as that at ambient temperature. A recent study by Wang and Orton (2008) concluded that the same design procedure in Eurocode 4 Part 1.1 may be used for both ambient and elevated temperature design CFT columns, provided modifications are used to reflect the changes in material mechanical properties and the influence of initial imperfections.
2.4 CFT CONNECTIONS

There are two main types of connections that may be used to connect beams to concrete filled tubular CFT columns, simple connections and moment-resisting connections. Simple connections are normally assumed to give vertical support but to provide only limited restraint against rotation: these connections are assumed to be able to rotate without damage. However, moment connections are those that are assumed to give vertical support, to provide a substantial degree of restraint against rotation and to develop some moment capacity. A selection of some common examples is shown in Figure 2.3 and Figure 2.4 (Hicks and Newman 2002).

The most common arrangements for beam-to-column joints make use of bolted connections via attachments welded to the faces of the hollow section column. By far the most common connection of this type is the fin plate connection (Figure 2.3(a), using a flat plate welded to each column face. For RHS columns, an alternative is the web cleat connection (Figure 2.3(b)), using single angle sections, or T-sections, welded to the column face. The use of double angles is a further option and provides greater capacity than a single angle would. However, if double welded angles are used, the difficulty of site joining should be considered since it may be difficult to slot the web of the beam into the gap formed by the two angles. An increasingly popular option for CHS or RHS columns is the use of the reverse channel connection (Figure 2.3(c)). In fact, the results of a recent study by Li (2012) have demonstrated that reverse channel connection with flush/extended endplates can develop substantial rotational stiffness and bending moment capacity to qualify as semi-rigid and moment resisting joints. Simple steel connections using flexible end plates (Figure 2.3(d)) or double angle cleats, which are bolted direct to the column, are also possible with RHS columns. These joints use either expanding bolt types, such as Hollobolt, or fully threaded bolts in tapped holes produced by the Flowdrill system.
Rigid or semi-rigid moment connections (Figure 2.4) are feasible with all types of hollow section column. These may use flange plates (Figure 2.4a&amp;b) or beam stubs (Figure 2.4(c)), which are usually of the same section as the beam being connected. Through-plated connections are another popular type of moment connection (Figure 2.4(d)); this is similar in appearance to the fin plate connection but has slots in the column to allow a single plate to be taken through it. In almost all cases, moment connections are more expensive than simple connections, but the extra cost of the connection can be more than offset by savings in beam sizes, or by provision of more usable floor space.
Research studies on the performance of joints to hollow section columns at ambient temperature date back many years. For example, White and Fang (1966) carried out experiments on fin plate, T-stub and double web cleat connections to RHS columns. Astaneh and Nader (1990) studied the performance of T-stub connections to RHS columns. Sherman and Ales (1991) and Sherman (1995) studied a large number of simple framing connections of I beam to RHS columns subjected to shear only. Again in 1995 Dawe and Mehendale investigated the performance of T-stub connections. France et al. (1999) conducted a series of experiments to investigate the performance of end-plate connections to concrete-filled and unfilled SHS columns. The results obtained from tests were presented in a series of three papers (France et al. 1999a, 1999b and 1999c). Wang (2009) et al. conducted an experimental program of bolted moment connection joints of circular or square concrete filled steel tubular (CFST) columns, and H-shaped steel beams using high-strength blind bolts. Wang et al. (2009a) investigated the static performance and failure modes of four blind bolted connections and Wang et al. (2009b) investigated another four bolted moment-resisting connections under simulated seismic loading conditions. Elghazouli et al. (2009) carried out seventeen
monotonic and cyclic tests to examine the behaviour of blind-bolted angle connections between open beams and tubular columns. Malaga-Chuquitaype and Elghazouli (2010) carried out another ten monotonic and cyclic tests to examine reverse channel connections using angles bolted at top, seat and web.

From the outcomes of these research studies, the behaviour of different types of connection to CFT column can be summarised as follows;

- The fin plate connection is simple to fabricate and allows easy site installation. However, it is a simple connection with very little stiffness and bending moment capacity. In addition, its performance under extreme loading may not be sufficient. Web cleat connections are similar in principle to the fin plate detail but can be entirely bolted.

- T-stub connection does not offer much greater stiffness and resistance than fin plate connection, but it can be more expensive to fabricate. The failure modes that were observed during the testing of T-stub connections were bolt shear fracture, tee stem yielding, tee flange yielding, bearing failure, tee stem fracture, and weld fracture.

- End plate connections offer great connection stiffness and strength to qualify in semi-continuous design. The problem with using this system in open beam to tubular column connection is that blind bolting is required due to inaccessibility to connect bolts to closed tubes.

- Reverse channel connections make use of the good performance of moment-resisting connections such as flush and extended end-plate connections, and also are able to use reliable connection methods without complicated fabrication and erection on site. The channel web is the dominated component in the reverse channel connection behaviour. The endplate thickness is also a key component. The failure modes that were observed during the testing of reverse channel connection were flange/web junction shear failure, bolt pull-out and horizontal endplate shear failure.
From this quick review, it is also not difficult to discern that these investigations have concentrated on connections to CFT column at ambient temperature, rather than at elevated temperature.

Recently, there have been a small number of researches on the fire performance of connections to both filled and unfilled tubular columns. Ding (2007) carried out 10 fire tests on structural assemblies in order to study the behaviour of steel beam to concrete filled tubular (CFT) column assemblies in fire using different types of joints. Jones (2008) has recently carried out some tests on fin plate and reverse channel connections to both filled and unfilled tubular columns. These research studies have provided experimental insight into structural behaviour of reverse channel connection in fire. They will be presented in more details in section 2.6.

As can be seen, it is clear that much research is still necessary to conduct more extensive research studies to improve understanding of the behaviour of joints to CFT columns in fire.

2.5 BEHAVIOUR OF RESTRAINED BEAMS IN FIRE

If a simply supported, axially unrestrained, laterally restrained steel beam is attacked by fire from underneath, failure of the beam happens when a plastic hinge mechanism is created, just as at ambient temperature. A simply supported beam without lateral restraint may fail due to lateral torsional buckling with reduction of the strength and stiffness. Again, this is similar to behaviour at ambient temperature. However, restraints to the steel beam have great effect on the beam structural behaviour in fire.

A longitudinally restrained beam under fire acts totally differently from a beam without longitudinal restraint. Wang (2002) has presented a qualitative explanation of the performance of longitudinally and rotationally restrained beam under fire as shown in Figure 1.1 of Chapter 1. According to the development of axial force in the beam, the behaviour of a restrained beam during heating can be divided into 3 main stages. In stage I, compressive axial force is induced in the beam by thermal expansion and increases with temperature elevation. In stage II the compressive force in the beam begins to decrease as a result of reduced stiffness of the beam and local buckling of the compression flange, until it
becomes to zero. In stage III, the axial force in the beam goes into tension.

In 2002, The University of Manchester carried out a series of tests on sub-frames (Liu et al. 2002) to determine the influence of catenary action and the axial forces developed on ultimate survival of the beams in fire. The experimental programme examined in detail the role of the connections and axial restraint in affecting the fire resistance of a steel beam when subjected to fire. Two different types of beam-to-column connection were examined, flush end-plate connection and web cleat connection, while the whole sub-frame assembly was subjected to various levels of loading. Additional horizontal restraints were applied at the level of the beam in order to replicate the restraining effect from the adjacent parts of the complete frame. Based on the tests results of Liu et al. (2002), the behaviour of the beam under the influence of connection and axial restraints became clearer. When the beam temperature starts to rise, the top flange stays cooler than the rest of the section, causing a downward thermal bowing. A hogging moment and an axial compressive force are induced along the beam. The connection moment increases and the mid-span moment decreases until there is either a reversal in temperature gradient or material yielding near the connection under a combination of high bending moment and axial compression force. This in general happens when the bottom flange temperature is in the range 450–600°C. The connection moment then starts to drop gradually and the mid-span moment increases. As the temperature rises and material yields further, the loss of bending strength makes it difficult to support the loading and runaway starts at the beam’s limiting temperature. The axial restraint then reacts quickly; the compression force drops rapidly and changes into a tension force. As catenary action takes place, the rate of run-away is slowed.

By using ABAQUS Yin and Wang (2004) examined the large deflection behaviour of steel beams at elevated temperatures with different idealised elastic, and elastic-plastic, axial and rotational restraints at the ends. A particular importance of this research was the behaviour of axially restrained steel beams in catenary action. After validating the capability of ABAQUS against available experimental results of fire tests on restrained steel beams, Yin and Wang (2004) showed the results of a numerical parametric study. The parameters investigated included beam span, uniform and non-uniform temperature distributions, different levels of applied load, and different levels of axial and rotational
spring stiffness at the beam ends and the effect of lateral torsional buckling. They concluded that if a steel beam is reliably provided with some axial restraints, catenary action will occur and will enable the beam to survive very high temperatures without a collapse. They noticed that the beam survival temperatures were only slightly affected by temperature distribution, rotational restraint or whether or not the beam would experience lateral torsional buckling. The level of axial restraint was the most important factor. The higher the axial restraint, the smaller the beam deflection, which is favourable for integrity of the fire compartment in which, the restrained beam is located. However, a higher axial restraint stiffness will also exert a larger catenary force on the structure adjacent to the beam. In addition, since the beam’s restraint comes from the adjacent structure, a higher axial restraint stiffness also means that the adjacent structure to the beam should be stiffer. Therefore, the exploitation of catenary action in realistic applications will involve careful consideration of the adjacent structure to the beam, including connections, and their design will be a compromise between the demands of providing restraint stiffness and resistance to the beam’s catenary forces, and the allowable beam deflection. If the beam’s large deflections do not become a design restriction, provided the adjacent structure has sufficient resistance to the beam’s catenary forces, they should be designed as flexible as possible to reduce the beam’s catenary forces.

Yin and Wang (2005a&b) further developed simplified methods to predict beam lateral deflection and axial force developments in axially restrained beams. Although these simplified predictive methods have been compared with numerical simulation results using ABAQUS, due to the idealistic nature of the assumed connection behaviour, application of their methods to practical design is yet to be confirmed. A main part of this research is to examine restrained beam behaviour when using realistic connections. In particular, how the connections may be changed to improve the beam’s survival temperature.

The research studies of Li and Guo (2008) are similar to the aforementioned research studies and came to similar conclusions during the heating phase, Li and Guo (2008) went farther and described the behaviour of the restrained beams during the cooling phase. They concluded that after the fire went out, a larger tension force was produced
in the restrained steel beams by contraction as the temperature decreased. Larger is the axial stiffness of the restraint; larger is the tension force that is induced. According to the results from the experiments of Li and Guo, the stiffness of the axial restraint plays an important role in the behaviour of restrained steel beams subjected to heating and cooling in a fire. More detailed investigation of cooling behaviour of steel beams connected to CFT columns and how the level of axial restraint affecting beam and connection behaviour during cooling is one of the main objectives of this research.

Analysing axially restrained steel beam behaviour in fire is a complex and challenging task. In a demonstrative study, Gillie (2009) enumerated the differences between small deflection analysis of beam behaviour at ambient temperature and large deflection behaviour of beam behaviour at elevated temperatures. Table 2.1 presents his main findings.

Two sets of independent results were used for validation study of this research. The benchmark exercise of Gillie (2009) will be repeated in chapter 4 of this thesis as partial validation of the numerical modelling used in this research.

Compared to other reported studies on the behaviour of restrained steel beams in fire, this research will incorporate realistic connection behaviour.
### Table 2.1: Comparison between the analyses processes for ambient and high-temperature structural design (Gillie 2009)

<table>
<thead>
<tr>
<th>Ambient temperature structural design</th>
<th>High-temperature structural design</th>
</tr>
</thead>
<tbody>
<tr>
<td>- At ambient temperature the “actions” on a structure typically result from a combination of wind and gravity loading. Such actions are forces and are (or can reasonably be assumed to be) non-varying when estimating strength.</td>
<td>- Since not all parts of a structure heat at the same rate, and because structural elements expand when heated, stresses are produced.</td>
</tr>
<tr>
<td>- The stresses in a structure at ambient temperature may be considered constant for each load case and it is straightforward to design for sufficient strength.</td>
<td>- The inter-play between thermal expansion, restraint to this expansion and the large deflections commonly present in fire conditions, also results in stresses within structural members varying during a heating–cooling cycle. A further complication is that heating and cooling will not occur simultaneously in all parts of a structure.</td>
</tr>
<tr>
<td>- Simplifications may be made as a result of the most commonly used structural materials being very stiff. This means that deflections can be considered to remain small and geometric non-linearity can be neglected in analyses.</td>
<td>- Thermal expansion also frequently causes large deflections to be present in heated structures. Large deflections do mean it is necessary to account for the effects of geometric non-linearity in structural analyses if accurate results are to be produced.</td>
</tr>
<tr>
<td>- It is usually possible to assume either linear elastic or rigid-plastic material behaviour, further simplifying the analysis process by removing the difficulties of handling material non-linearity in calculations.</td>
<td>- High temperatures also affect structural materials’ mechanical properties with key factors being the loss of linearity, strength, modulus and a clear yield point. Thermal expansion also frequently causes large deflections to be present in heated structures.</td>
</tr>
</tbody>
</table>
2.6 BEHAVIOUR OF JOINTS TO CFT COLUMNS IN FIRE

It is now clear that it is important to understand the behaviour of steel beam-to-CFT column connections in fire in order to improve the resistance of robustness of the CFT structure in fire. Although research on connection behaviour at ambient temperature is abundant, research on connection behaviour in fire is at a relative early stage. In particular, most of existing research studies on connection behaviour (for example, Leston Jones 1997, Al-Jabri 1998, Al-Jabri et al. 2005, Wang et al. 2007, Qian et al. 2008, Amir and Mahmood 2008, Chung et al. 2010) in fire mimic the ambient temperature research methodology by focusing on changes in moment-rotation characteristics of connections at high temperatures.

It is most recently that the research focus has changed to also include the effects of axial force in connections. In particular, the Universities of Sheffield and Manchester have recently conducted a joint research programme with the aim of investigating the tying capacity and ductility of steel connections at elevated temperatures.

The University of Manchester conducted 10 fire tests on structural assemblies (Wang et al. 2011) in order to study the behaviour of steel beam to open column assemblies in fire using different types of joints. Figure 2.5 shows the test assembly. In total, 10 tests were carried out in a Rugby style arrangement. Each tested specimen consisted of two columns and one beam jointed together by two joints. Five different joint types were investigated: fin plate, flexible end plate, flush endplate, web cleat and extended endplate. The cross-sections of the beams in all tests were the same (UB 178 x 102 x 19). Two types of column sections were used. The first one was S355 universal column UC 254 x 254 x 73 (test1-test5) and the second one was S275 universal column UC 152 x 152 x 23 (test6-test10), representing different amount of restraint to the steel beam. Two 40kN concentrated loads generated by jacks were applied on the beam. These loading jacks were 0.6m away from the columns. The columns were prevented from moving at the bottom but free to move in the vertical direction at the top. The top flange of beam was covered by fire protection and restrained in the lateral direction by a light weight steel truss. The test results showed that with respect to joint robustness, the fin plate and flexible end plate connections are more brittle compared with the other three connections, so that the fin plate and flexible end plate connections could not develop.
substantial catenary action before connection failure. The failure modes include weld fracture; bolt shearing, beam web shearing and end plate shearing. Although flush end plate connection had high stiffness, the bolts failed by stripping, achieving very low ductility to allow substantial catenary action. The stiff extended end plate connection and flexible web cleat connection did not fail and developed suitable catenary action due to high ductility of end plate and web cleats respectively.

Because of the complexity in connection behaviour in fire when variable combinations of axial force and bending moments are involved, the flexibility of the component based connection characterization method is particularly appealing. Essentially, in the component based method, a connection is represented by an assembly of springs each under a particular single action (axial or shear), such as shown in Figure 2.6 for a flexible endplate connection (Hu et al. 2009). These components may be arranged to give complex interactions between axial force and bending moment in the connection at high temperatures. For connections between open steel sections at ambient temperature, the component based method has been developed to a very detailed level and the methodology is encoded in Eurocode 3 Part 1.8. Recent research studies by various researchers (Leston-Jones 1997, Simoes da Silva et al. 2001, Al-Jabri 2005, Sarraj et al.
2007, Hu et al. 2009 and Taib and Burgess 2011) indicate that the ambient temperature component behaviour is largely useful to quantifying elevated temperature behaviour.

**Figure 2.6: Application of the component based method to flexible endplate connection (Hu et al. 2009)**

Compared to connections between open steel sections, research on joints between steel beams and concrete filled tubular columns in fire is scarce. The most relevant studies are reported by the author’s research group led by Wang.

Among these research studies, Ding and Wang (2007) carried out 10 fire tests on structural assemblies in order to study the behaviour of steel beam to concrete filled tubular (CFT) column assemblies in fire using different types of joints. The test arrangement was similar to that used by Wang et al. (2011) as shown in Figure 2.5. The joint types included fin plate, end plate, reverse channel and T-stub. In each test, loads were applied to the beam and then the structural assembly was exposed to the standard fire condition in a furnace while maintaining the applied loads. In total, 10 tests were carried out. In eight of the 10 tests, fire exposure continued until termination of the fire test, which was mainly caused by structural failure in the joints under tension when the beam was clearly in substantial catenary action. In the other two tests (one using fin plates and one using reverse channels); fire exposure stopped and forced cooling started when the beam was near a state of pure bending and just about to enter into catenary action.
The results of the experiments indicate that using reverse channel connections was able to allow the beams to develop substantial catenary action so that the final failure times and beam temperatures of the assemblies were much higher than those obtained by assuming the beams in pure bending. At termination of the tests, the beams reached very high deflections (about span/5); even then failure of the assemblies always occurred in the joints. Therefore, to enable the beams to reach their full potential in catenary action, the joints should be made to be much stronger and achieve more deformation. This conclusion by Ding (2007) is the most important justification for the research carried out by the author. Due to time limit, Ding was unable to carry out detailed numerical simulations of the tests. In the next two chapters, the author will present a more detailed description of these tests and comprehensive comparison between numerical simulations and the test results to validate the numerical simulations.

Another conclusion from the fire tests of Ding and Wang (2007) was the drastic difference in behaviour, in terms of residual deflection and axial tensile force in the connection, of the two cooling tests. By controlling the starting temperature from which the beam cools down, it is possible to significantly reduce the residual tensile force in the connection so as to avoid connection fracture, such as those shown in Figure 1.2 in chapter 1 from the Cardington fire tests. More detailed investigation of cooling behaviour of steel beams connected to CFT columns will form another objective of this research.

For efficient and accurate representation of joint behaviour, the component-based model has been accepted as the most effective approach. As part of research on joints to CFT columns, Jones and Wang (2008) conducted an experimental study into the behaviour of welded fin plate connections to both hollow and concrete filled tubular (CFT) columns under shear. The experiments were performed at both ambient and elevated temperatures with the aid of an electric kiln. The test results showed that the observed failure modes included fracture of the fin plate and tearing out of the tube around the welds. Concrete in-fill was observed to significantly increase the strength of connections over empty specimens, and circular column specimens were observed to exhibit greater strength than similarly proportioned square columns. Jones (2008) also conducted a limited number of tests and numerical validation for reverse channel to
CFT connections loaded in shear at both ambient and elevated temperatures. Jones found that in general, compared to the fin-plate tests the reverse channel specimens exhibited higher resistance to shear and bending load.

In this research, due to a lack of component behaviour models, the component-based model will not be used.

2.7 BEHAVIOUR OF STRUCTURAL FRAMES IN FIRE AND FIRE INDUCED PROGRESSIVE STRUCTURAL COLLAPSE

Obviously, element failure could occur as a result of any of several extreme loading events on buildings, including strong earthquakes, blast, vehicle impact, fire, or similar incidents. Progressive collapse is caused by a series of structural element failures due to unforeseen large internal loads that exceed the elements’ load bearing capacities. The internal loads may be generated by sudden geometry change of the structure (Marjanishvili 2004) or a structural behaviour mechanism that has not be considered in the initial design. The collapse of WTC 7 represents the first known instance of the fire induced progressive collapse. Figure 2.7 shows the consequence of joint failure, leading to progressive structural collapse, in the World Trade Center building 7 failure. In the Twin Towers disaster, seated connections were used to connect key columns 79-81 to the beams. These connections were designed as shear connections. However, it is believed that during the fire attack on this building, axial forces were generated in these connections due to thermal elongation of the connected beams being restrained by the columns. The incapability of these seated connections to resist these axial forces, which were not part of the design load, was believed to have initiated progressive collapse of the building.

From the study of WTC7 collapse, NIST (2008) (National Institute of Standards and Technology) gave factors that could have prevented the progressive collapse. One of the most particular concern are the effects of thermal expansion in buildings with one or more of the following features: long span floor system, connections not designed for thermal effects, asymmetric floor framing, and composite floor system. More robust connection and framing system should be developed to better resist the effect of the thermal expansion of the structural system. Fundamental to the method to improve the
robustness of joints is to form alternative load paths. Catenary action represents an effective alternative load path and how realistic joints may be designed to allow full development of catenary action in beams is an important objective of this research.

![Figure 2.7: Seat connection and simulated failure of support to columns 79-81 of WTC7 building (NIST 2008)](image)

2.8 NUMERICAL SIMULATIONS OF JOINTS AND STRUCTURAL ASSEMBLIES IN FIRE

Numerical simulation started being used as a way to overcome the lack of experimental results; The use of FEM to study connection behaviour started in early 1970s, as the application of computers in solving structural problems became evident.

The finite element method (FEM) provides an attractive means of investigating the behaviour of beam-to-column joints in more detail than experimental tests would usually allow. This is for many reasons;

- Tests can only be performed on a limited number of specimens;
- Due to the limitations of the testing facilities, quite often tests are restricted to small-scale specimens;
- Tests generally performed in enclosed furnaces, which provide limited access for direct viewing and measuring equipment.

Under these circumstances, finite element analysis provides a useful method of investigating the test behaviour, with a view to understanding failure mechanisms for the following three reasons;
• Finite element models allow the understanding of local effects which are difficult to measure accurately physically;
• Finite element models are economical;
• Finite element models can be used to generate extensive parametric studies.

Liu (1996) was the first to attempt to use FEM in modelling connection behaviour at elevated temperature (Al-Jabri 2008). He developed a finite element model (Figure 2.8), based on an eight-noded isoparametric element, for the study of the behaviour of steel structure under the effect of fire. In addition to the eight-noded shell element, which was used to discretize the flanges and webs of the beam, column, stiffeners and end-plate, a beam element having special characteristics was used to simulate the behaviour of bolts and the contact 'link' between end-plate and column flange. The model included the consideration of the material plasticity and deterioration with temperature, non-uniform thermal expansion across a section and large deformations at very high temperatures. Analyses were undertaken using time steps. Comparisons were made with the test data obtained from real fire tests on both steel beams with different support and loading conditions, and with beam-to-column connections. Both the simulated deflection-time curve and fire resistance period were found to be in good agreement with the test evidence. Liu (1996) mentioned that the advantage of the three-dimensional discretization modeling is that it allowed a more extensive study of structural behaviour to be undertaken. Having verified the modeling technique, more sophisticated modeling of fire resistance of other components and indeed complete assemblies, e.g. the composite effect between steel beam and concrete slab, and the effect of flexible connections on fire endurance of continuous construction can be carried out. Again the three-dimensional mathematical model “FEAST” which was developed to simulate the response of steel structures in the event of a fire (Liu, 1996), had been further developed in 1998 to model the behaviour of steel/concrete composite connections at elevated temperatures. The model was based on a finite element procedure with special consideration of bolts, shear studs and concrete behaviour. The temperature-rotation characteristic of the connections is produced at various applied bending moments. The mathematical model was compared with a series of fire tests carried out at the Building Research Establishment, UK (Lennon, 1995). The comparison was found encouraging.
Further comparison between the behaviour of composite and non-composite connections was also presented.

Figure 2.8: Mesh discretization and deformed shape of the connection (Liu 1996)

In 1999, Liu used finite element modeling to study the structural behaviour in fire of simple sub-structured frames consisting of unprotected steel I-beams and columns, which connect through end-plate bolted connection. Flush end-plate and extended end-plate connections were considered. Various parameters which affect the behaviour of end-plate type connections were examined, such as bolt size, number of bolts and end-plate thickness. Furthermore Liu presented a 3D mathematical model based on the finite-element method to simulate the structural response of steel and steel/concrete composite connections to predict their moment-rotation-temperature characteristics. Liu considered the beam failure in the course of a fire when the mid-span deflection exceeded L/30 and the rate of increase of deflection was excessively high. Liu concluded that with a similar applied load ratio, the beams with flush end-plate connections would have similar fire resistance as if they were simply-supported. However the beams with extended end-plate connections were benefited to a larger extent. The limiting temperature could be increased by as much as 45°C relative to simply-supported beams. Liu’s research was limited to small deflection behaviour.

El-Houssieny et al. (1998) studied the moment rotation curve, bolt forces and stresses for semi-rigid extended end plate connections to develop simple prediction equations to contribute to the understanding of the behaviour of different connection components at elevated temperatures. The behaviour of the connection was studied for both normal and temperature conditions. A three-dimensional finite element model was developed to simulate the connection behaviour. Homogeneous thermal gradients across the cross-section of the connection were assumed. The end-plate and column flange were
modelled by solid elements, whereas the beam and stiffener were modelled using shell elements. Each bolt was assembled using beam elements. Close agreement was obtained with experimental work and subsequent parametric studies were conducted to investigate the influence of connection behaviour at normal and elevated temperatures. This research concentrated on moment-rotation characteristics of connections and there was no consideration of the effects of restraints and very large deflections in beams.

Al-Jabri et al. (2005) performed a study of the behaviour of flush end-plate bare-steel connections at elevated temperatures using the general purpose finite element software ABAQUS. The finite element model was used to establish the moment–rotation characteristics of the connections under the combined loading of a concentrated force and elevated temperature. The connection components were modelled using three-dimensional brick elements as shown in Figure 2.9, while contact between the various components was modelled using Coulomb friction. Materials for steel members and connection components were considered to behave nonlinearly. Degradation of steel properties with increasing temperature was assumed in accordance with EC3 recommendations. Finite element results and experimental tests conducted on flush end-plate connection in fire conditions were compared, and these showed good agreement. The obtained simulated failure modes and moment–rotation–temperature characteristics of the connections compared well with the experimental data. This proved that the FE technique is capable of predicting connection response at elevated temperatures to an acceptable degree of accuracy.

Figure 2.9: 3-D FE model and predicted deformation of the connection (Al-Jabri et al. 2005)
Sarraj et al. (2007) developed a three-dimensional (3-D) FE model of a fin plate connection using the ABAQUS software in order to analyse and understand the behaviour of such a connection at ambient and elevated temperatures. The starting point for this model was a simple plate with a bolt bearing against a hole (Figure 2.10). This model was then developed to form the single lap joint. Ultimately, the entire fin plate connection was assembled and modelled using a series of lap joints in which one plate was the fin plate, and the second plate was the beam web. The three main parts of the fin plate connections; the beam, fin plate and bolts were modelled using eight-node continuum hexahedral brick elements. The models incorporated non-linear material properties for all the connection components, geometric non-linearity and contact behaviour. Contact elements have been used both at the bolt–hole interface and also at the surface between the web of the beam and the fin plate, taking into consideration friction between the surfaces. A friction coefficient of $\mu = 0.25$ was adopted for all the contact surfaces. The connection model analysed through the elastic and plastic ranges up to failure. A comparison between available experimental data at ambient and elevated temperatures and other analytical results showed that the model had a high level of accuracy. From the finite element model results Sarraj et al. (2007) concluded that a full 3-D solid model incorporating contact and nonlinear material properties can be used to model accurately the fin plate shear connection behaviour using ABAQUS/Standard software. Bolt shear and bending, and plate and web bearing were observed as failure modes of the fine plate connection. When the connection model was extended to include an attached beam, the beam experiences large tensile force when exposed to fire.
Yu et al. (2008) explored the use of an explicit dynamic solver to analyse the behaviour of bolted steel connections. The general commercial program ABAQUS was used to model two types of connections, flush endplate connection and web cleat connections. Yu et al. (2008) suggested that, for typical steel connections, a loading duration of 0.1–1s in the explicit dynamic analysis should ensure a quasi-static response. The effects of varying mesh sizes representing the bolts and the connected plates were studied. The responses were generally not very sensitive to the mesh size unless extremely coarse meshes were used. Finally explicit dynamic analysis was used to simulate three sets of connection tests at ambient and elevated temperatures. Yu et al. (2008) concluded that the analytical results were found to be in close agreement with the test results.

The explicit method may used to obtain solutions for static problems, but in this case large numbers of increments are needed, even though the results may not be as accurate as the static solution of ABAQUS/Standard. In most cases, to obtain reasonable results for static problems using ABAQUS/Explicit, a minimum of 300 000 increments are required (Sarraj et al. 2007).

Due to the huge requirement on computational resources (both time and storage) and unstable structural behaviour when using the ABAQUS/Explicit solver, in this research, the author decided to use the ABAQUS/Static solver.
Hu et al. (2008) created a three dimensional numerical model for a flexible end plate connection (Figure 2.11), using explicit dynamic analysis in the ABAQUS finite element code, in order to investigate its resistance and ductility at ambient and elevated temperatures. This model started with the creation of individual components such as bolts, endplates, beams and columns, and then assembled these components. All these components were modelled using eight-node continuum hexahedral brick elements, and a small number of cohesive elements were used in the heat affected zone (HAZ) where the failure of endplates was seen to occur. In comparison with experimental test data, a good correlation with the finite element analysis was achieved and the method was suitable to study the tying resistance and ductility for simple steel connections with various dimensions at different temperatures.

Figure 2.11: FE model for a flexible end plate connection (Hu et al. 2008)

Mao et al. (2009) investigated the fire response of steel semi-rigid beam-column moment connections made with H-shape beam and H-shape column using the general purpose finite element software ANSYS. The 8 nodes Solid185 element and Solid70 element are adopted to process structural and thermal analysis. The numerical model was verified by the full-scale fire tests implemented in the building fire laboratory centre of the Architecture and Building Research Institute (ABRI) in Taiwan, and its results were found to agree well with experimental results.
Yu et al. (2009) used the general-purpose program ABAQUS to model web cleat connections (Figure 2.12) under tying force in fire. Tests were performed on web cleat connections at various temperatures, while subjected to different combinations of tension and shear loading. The explicit dynamic analysis was adopted. A good correlation between the numerical results and experimental data demonstrated that the investigation of the behaviour of the connection, with some proposed modifications to the general finite element model, showed that finite element analysis can help to interpret the test results and expand the test observations to other similar applications.

Figure 2.12: FE model for web cleat connections (Yu et al. 2009)

Qian, et al. (2009) conducted a series of finite element simulations using the commercial finite element analysis (FEA) software package MSC to rationalise the results of six extended end-plate beam-to-column joints cruciform tests at elevated temperatures. The beam, column and end plate modelled by quadratic thick shell elements. In addition to the eight-noded shell elements, the connecting bolts are simulated by three sets of springs, representing both axial tension stiffness and a two-dimensional shear stiffness. As result of very reasonable agreement with the test results Qian, et al. (2009) concluded that FE models of the order of complexity used were able to predict the steel joint behaviour accurately.
Selamet and Garlock (2010) used ABAQUS/Standard to investigate possible ways to modify the single plate shear connection for improved fire performance. They conducted that by using finite element model validated by a full-scale frame Cardington building test (Garlock and Selamet 2010). It was concluded that significant improvements in the behavior of single plate connections can be achieved by using any of the following modifications: adding a doubler plate to the beam web, matching the single plate thickness to the beam web thickness, using a larger distance from the bolt-hole centerline to the beam web thickness, using a larger distance from the bolt-hole centerline to the beam web thickness, using a larger distance from the bolt-hole centerline to the beam web thickness, increasing the gap distance between the end of the beam to the connected member.

Chung et al. (2010) used a general-purpose finite element implicit solver, ABAQUS/Standard, to perform 3-D nonlinear FE simulations for two full-scale beam-to-column moment welded connection (Figure 2.13) specimens tested at elevated temperatures according to the standard ISO-834 fire. For the 3-D nonlinear FE structural analyses in ABAQUS/Standard, both material and geometric non-linearities were considered. The high-temperature stress-strain relationships of SN490B steel, SN490C-FR fire-resistant steel, and S10T bolts obtained from the high-temperature tensile coupon tests, as well as the Von Mises yield criterion and the associative flow rule, were adopted to model the material nonlinearities of the two specimens. Geometric nonlinearity was also taken into consideration by activating the optimal parameter "NLGEOM" in ABAQUS/Standard to capture the large deformation and local instability effects in the 3-D FE models of the two specimens.
Diaz et al. (2011) used a full three-dimensional ANSYS finite element model of steel beam to column bolted extended end-plate joints to obtain their behaviour. The model includes: contact and sliding between different elements; bolt pre-tension; and geometric and material non-linearity. The model was calibrated and validated by comparing the moment–rotation curve of the joint with experimental results found in the literature and with the model proposed by Eurocode 3.

Recently Dai et al. (2010) presented the results of a simulation study of 10 fire tests on restrained steel beam column assemblies using five different types of joints: fin plate, flexible endplate, flush endplate (Figure 2.14), web cleat and extended endplate. Details of the simulation methodology for achieving numerical stability and faithful representation of detailed structural behaviour, and compare the simulation and
experimental results, including joint failure modes (Figure 2.15), measured beam axial forces and beam mid-span deflections were presented. Good agreement between ABAQUS simulations and experimental observations confirmed that the finite element models developed through the ABAQUS/Standard solver are suitable for predicting the structural fire behaviour of restrained structural assemblies with realistic steel joints undergoing different phases of behaviour in fire, including restrained thermal expansion and catenary action in the beams. The 3D finite element model adopted to simulate joints to CFT column in this research is similar to Dai model.

Figure 2.14: Typical FE model adopted in numerical modelling (Dai et al. 2010)

Figure 2.15: Observed deformation patterns of flush endplate joint (Dai et al. 2010)

Table 2.2 summarizes the studies focused on the numerical simulation of joints and structural assembly in fire.
<table>
<thead>
<tr>
<th>Author</th>
<th>Year</th>
<th>Software package</th>
<th>Parameter</th>
<th>Methodology</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lui</td>
<td>1996-1999</td>
<td>FEAST</td>
<td>Temperature-rotation characteristic</td>
<td>Static analysis using 2D modelling, using eight-noded shell elements used to model the beam, column, stiffeners and end-plate, a beam element to simulate the behaviour of bolts and the contact.</td>
</tr>
<tr>
<td>El-Houssieny et al.</td>
<td>1998</td>
<td>N/A</td>
<td>Moment rotation stiffness of extended end plate connections</td>
<td>Static analysis using 3D modeling, using solid elements to model the end-plate and column flange, whereas shell elements for the beam and stiffener. Each bolt was assembled using beam elements.</td>
</tr>
<tr>
<td>Al-Jabria et al.</td>
<td>2005</td>
<td>ABAQUS</td>
<td>Flush end plate connections</td>
<td>Static analysis using 3D modelling using three-dimensional brick elements</td>
</tr>
<tr>
<td>Sarraj et al.</td>
<td>2007</td>
<td>ABAQUS</td>
<td>Fin plate connections</td>
<td>Static analysis using 3D modelling. The beam, fin plate and bolts were modelled using eight-node brick elements.</td>
</tr>
<tr>
<td>Yu et al.</td>
<td>2008 &amp; 2009</td>
<td>ABAQUS</td>
<td>Flush endplate and web cleat connections</td>
<td>Explicit dynamic analysis eight-node brick elements.</td>
</tr>
<tr>
<td>Hu et al.</td>
<td>2008</td>
<td>ABAQUS</td>
<td>Resistance and ductility of flexible end plate connection</td>
<td>Explicit dynamic analysis eight-node brick elements.</td>
</tr>
<tr>
<td>Dai et al.</td>
<td>2010</td>
<td>ABAQUS</td>
<td>Restrained steel beam column assemblies</td>
<td>Static analysis using 3D modelling using three-dimensional brick elements</td>
</tr>
</tbody>
</table>
From the presented overview it is clear that FE methods provide a reliable technique, which can efficiently be used in predicting the elevated-temperature behaviour of joints to an acceptable degree of accuracy and enable a wider range of parameters to be considered than would be the case with a laboratory-based investigation.

However, so far, all these existing numerical simulations of joint behaviour, except that of Dai et al (2010), have considered statically determinate structures in which the joint forces do not change with time and the effects of large deformation in the connected beam (if any) are not included.

Based on these aforementioned studies, Chapter 3 of this thesis will present the assumptions of the numerical model using ABQUS to simulate the behaviour of restrained structural subassemblies of steel beam to concrete filled tubular (CFT) columns and their joints in fire.

2.9 ORIGINALITY OF RESEARCH

In these aforementioned investigations, the majority of research work that has been carried out to study the behaviour of beam-column joints was focused on testing and modelling isolated joint configurations, where the effect of structural continuity and the presence of axial forces are usually ignored. Therefore, joints were mainly assumed to act in bending and the major research efforts were directed at establishing moment–rotation responses in the absence of axial force in the beam. Also, attention was focused on simulating joint response before very large deformations, so any inaccuracy in predicting joint behaviour during the very large deformation stage would often be overlooked. In contrast, in the fire tests conducted at the University of Manchester, the joint forces were variable throughout the fire exposure, as would be expected in realistic structures. This requires that the large deflection phase of structural behaviour be accurately modelled. Also, the existing research studies do not provide information on how to improve structural robustness, through modifying joint details, under fire conditions, both during heating and cooling.

Due to such limitations, this research will use ABAQUS software to develop and validate a three-dimensional (3-D) FE model to study the behaviour of restrained
structural subassemblies of steel beam to concrete filled tubular (CFT) columns and their joints in fire. Very large deflections will be included and both the heating and cooling stages will be considered. Afterwards, the validated model will be used to investigate the possible connection detailing of reverse channel connection for improved structural robustness in fire. An important part of the objective of this research is to study the overall structural behaviour of frames using CFT columns with realistic reverse channel joints. Finally, the research will examine the effects of cooling stage on the behaviour of restrained structural subassemblies of steel beam to CFT column, and how to minimize the risk of structural failure during cooling.

2.10 SUMMARY

This chapter has briefly summarized the advantages of CFT columns, performance of CFT columns at both ambient and elevated temperatures and the common connections to CFT columns. After that, this chapter introduced existing relevant studies of behaviour of restrained steel beams in fire and the importance of catenary action in improving the survival time of steel beams in fire. A review of existing relevant studies of the behaviour of joints in fire revealed that joints are critical members of the steel structure; however, few research studies have been reported on the behaviour of joints between steel beam and CFT columns in fire. Therefore, much research is still necessary to improve understanding of the behaviour of joints to CFT columns. The objective of this research is to fill some of the gaps in understanding the behaviour of joints to CFT column in fire particularly, reverse channel connections, and to identify means of detailing joints to CFT columns for improved structural robustness in fire during both heating and cooling.
CHAPTER 3 – NUMERICAL MODELLING OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMNS IN FIRE

3.1 INTRODUCTION

The aim of this research is to provide detailed information to structural performance of the joints and structural assemblies using concrete filled tubular (CFT) columns in fire during heating and the cooling stage. Therefore, this chapter will employ the general finite element software ABAQUS to numerically model the behaviour of restrained structural subassemblies of steel beam to CFT columns and their joints in fire. The simulations were conducted using 3-D brick elements to enable detailed structural behaviour to be obtained. For validation, this research compared the simulation and test results for 10 fire tests recently conducted at the University of Manchester (Ding and Wang 2007). Four different types of joints were used in the tests, including fin plate welded to the tubular wall, T-stub bolted to the tubular wall, reverse channel welded to the tubular wall, endplate bolted to the tubular wall. It was envisaged that catenary action in the connected beams at very large deflections would play an important role in ensuring robustness of steel framed structures in fire. Therefore, it was vital that the numerical simulations could accurately predict the structural behaviour at very large deflections. In particular, the transitional behaviour of the beam from compression to catenary action presented tremendous difficulties in numerical simulations due to the extremely high rate of deflection increase. To overcome this problem, a pseudo damping factor was introduced in ABAQUS simulations. It is important that this pseudo damping factor was not too high to render the simulation results inaccurate, but not too low so that its use to overcome numerical difficulty was made ineffective. How to select an appropriate pseudo damping factor will be presented in this chapter. As part of the validation study, sensitivity studies on finite element mesh, boundary conditions to the
test structures and gap size between the bolt and the plate were performed and their results will be presented in this chapter.

3.2 A BRIEF SUMMARY OF THE STRUCTURAL ASSEMBLY FIRE TESTS

In this section, a brief description of the 10 structural fire tests conducted at the University of Manchester (Ding and Wang 2007) will be provided. The main aim of these fire tests was to provide detailed experimental information to help quantify temperature fields in the joint region and structural performance of the joints and structural assemblies in fire. In the first 8 tests, the beam was loaded and heated to very high temperatures and very large deflections so that these fire tests provided information on joint behaviour over the entire range of steel beam behaviour during heating. The other 2 tests included a cooling phase and provided information on structural behaviour of the joints in cooling stage.

3.2.1 Testing specimens

The fire tests were conducted in the Fire Testing Laboratory of the University of Manchester and a total of 10 fire tests were performed. The standard fire exposure condition was followed in all tests. Eight of the 10 test specimens were tested to failure during heating. And in the other two tests, the test assembly was heated to temperatures close to the limiting temperature of the simply supported steel beam and then cooled down while still maintaining the applied loads on the beam. Each test assembly consisted of two concrete filled tubular (CFT) columns and a steel beam. The cross-section size of the beams in all tests was the same; being grade S275 universal beam section 178×102×19UB. Referring to Table 3.1, square hollow section (SHS) was used in 7 tests and circular hollow section (CHS) tubes in three tests. The overall dimensions of the SHS tubes were 200×200mm and the diameter of the CHS tubes was 193.7mm. Four different joint types were investigated: fin plate welded to the tubular wall, T-stub bolted to the tubular wall, reverse channel welded to the tubular wall, endplate bolted to the tubular wall). Grade 8.8 M20 bolts and nuts were used to connect fin plate to beam, end plate to reverse channel and T-stub connection to beam. A new blind bolting system
called Molabolts (www.molabolts.com) was used to connect the endplates and T-stubs to tubular wall.

Table 3.1: Summary of fire test specimens (Ding & Wang 2007)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Tube size</th>
<th>Thickness (mm)</th>
<th>Joint type</th>
<th>Applied load per jack (KN)</th>
<th>Design Load Ratio*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SHS 200</td>
<td>5.0mm</td>
<td>(fin plate) Plate 80x130x8</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>2</td>
<td>SHS 200</td>
<td>5.0mm</td>
<td>(bolted T) 133×102×13</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>3</td>
<td>SHS 200</td>
<td>12.5mm</td>
<td>(bolted T) 133×102×13</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>4</td>
<td>SHS 200</td>
<td>12.5mm</td>
<td>(reverse channel) 152×76 channel section 130 long, endplate 130x130x8</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>5</td>
<td>SHS 200</td>
<td>5mm</td>
<td>(reverse channel) 152×76 channel section 130 long, endplate 130x130x8</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>6</td>
<td>SHS 200</td>
<td>5.0mm</td>
<td>(extended endplate) Plate 260x130x8</td>
<td>60</td>
<td>0.5</td>
</tr>
<tr>
<td>7</td>
<td>SHS 200</td>
<td>5.0mm</td>
<td>(extended endplate) Plate 260x130x8</td>
<td>30</td>
<td>0.25</td>
</tr>
<tr>
<td>8</td>
<td>CHS 193.7</td>
<td>5.5mm</td>
<td>(reverse channel) 152×89 channel section 200 long, endplate 200x130x8</td>
<td>45</td>
<td>0.5</td>
</tr>
<tr>
<td>9</td>
<td>CHS 193.7</td>
<td>5.5mm</td>
<td>(fin plate ) Plate 90x140x8</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>10</td>
<td>CHS 193.7</td>
<td>5.5mm</td>
<td>(reverse channel) 152×89 channel section 130 long, endplate 130x130x8</td>
<td>30</td>
<td>0.5</td>
</tr>
</tbody>
</table>

* Defined as the ratio of the applied load to the load carrying capacity at ambient temperature of the simply supported beam without lateral torsional restraints with span
3.2.2 Testing method

The arrangement of the test specimen members was in the form of a complete ‘Rugby goalpost’ frame as shown in Figure 3.1. The steel beam was mainly unprotected. In order to simulate the heat-sink effect of the concrete slab in realistic structures, the top flange of the beam was wrapped with 15mm thick ceramic fibre blanket. The CFT columns were unprotected. The columns were restrained from lateral movement (so as to provide axial restraint to the beam) at the ends, see Figure 3.1, and were free to move in the longitudinal direction. Two transverse point loads were applied to the beam using two independent hydraulic jacks. In order to ensure that the loading jacks remained attached to the beam, each loading jack was inserted into a steel bracket, shown in Figure 3.2, which was then clamped on the top flange of the beam. It will be shown in the sensitivity study of this chapter that the steel bracket effectively provided lateral and torsional restraint to the steel beam. Also in order for the two loading jacks to avoid sliding towards each other during the beam’s large deflection and rotation, a steel bar (wrapped in insulation) was used to separate the two loading jacks.

The applied load on each jack was calculated to give a nominal load ratio of 0.5 for the steel beam. The nominal load ratio is defined as the ratio of the applied load during the fire test to the nominal load-carrying capacity of the beam with S275 grade steel at room temperature. The nominal load-carrying capacity of the beam was calculated by assuming simple joints at each end and with lateral torsional buckling.
Before fire exposure, loads were applied manually by hydraulic jacks on the beam at the two positions shown in Figure 3.2. After that the furnace temperature was increased to follow the ISO 834 standard fire (ISO 1980). Afterwards, the furnace was switched off to cool down the specimen. The temperatures at various locations, the horizontal reaction forces in the columns, displacements on the beam and the columns and the applied concentrated loads were measured continuously throughout the test. Attempts were made to maintain the applied loads throughout the fire test. However, as the rate of beam deflection increase became very high during the transition from flexural bending to catenary action in the beam, there were short periods when the applied loading failed to catch up with the deflection. Measurements were made at 10 second intervals.
3.2.3 Main experimental conclusions

The main relevant experimental observations were as follows:

- All the joints behaved well before the steel beam had reached its limiting temperature for flexural bending.

- It is possible for moderately complex joints to CFT columns (T-stub, reverse channel) to develop substantial catenary action, whereby the steel beams could survive higher temperatures than the limiting temperatures for flexural bending. The fin plate joint had little resistance and deformation capacity to develop catenary action in the beam. Among the few connections tested, the reverse channel connection appeared to have the best combination of important features: moderate construction cost, ability to develop catenary action, and extremely high ductility (rotational capacity) through deformation of the web of the channel. Thinner reverse channels (tests 4 & 5) would be preferable to thicker ones (test 8) as thinner ones would deform into a ductile mode (folding) whilst thicker ones may fracture in shear.
- The failure modes of the test specimens were always in the connection regions and they were verified by comparing the strength of different connection components with the appropriate applied loads at different test times.

- During cooling, considerable tension forces developed in the steel beam and the connections.

- Figures 3.3-3.6 show the recorded axial force developments and maximum deflections in the test beams from the different tests.

\[ \text{Beam bottom flange mean temp. (°C)} \]

![Figure 3.3: Beam axial forces – temperature relationships of tests 1-8 (Ding & Wang 2007)]
Figure 3.4: Beam axial forces – temperature relationships of tests 9 & 10 (Ding & Wang 2007)

Figure 3.5: Beam deflections – temperature relationships of tests 1-8 (Ding & Wang 2007)
3.3 DESCRIPTION OF THE FINITE ELEMENT MODEL

ABAQUS consists of two main analysis approaches: Abaqus/Standard and Abaqus/Explicit. The standard analysis is based on static equilibrium and characterized by simultaneous solution of a set of linear or nonlinear equations. For static nonlinear load-displacement problems, the Newton-Raphson iteration method (General static analysis) can be used until zero or negative tangent stiffness. At ambient temperature, the negative stiffness part of structural response may be dealt with using the Arc-Length method (Riks method) to achieve numerical convergence. However, the Riks method is not useful when dealing with structural fire response because here the applied load is maintained but the structural temperatures are changed. The other disadvantage of static analysis is that in bolted connections, the bolt surface and the inner edge of the hole must be carefully brought into initial contact; otherwise, the model will encounter zero stiffness, and thus numerical singularity. For simple contacts with obvious load-transfer routes, this problem is easy to be overcome. However, for connections with multiple bolts and subjected to complex loading, the pattern of initial slip might not be so obvious. Moreover, in a bolted connection, one bolt generally requires the definition of
four contact pairs: the contact between, the nut and the plate surfaces; the head and the plate surfaces; the bolt shank and the hole surfaces of the two connected plates. In this case it is almost impossible for the static analysis solver to converge (Yu 2008). However, as will be described later in section 3.3.5, ABAQUS/Static solver offers an automatic stabilization mechanism to handle such problems.

The ABAQUS/explicit solver employs dynamic analysis. This analysis algorithm strongly depends on the displacements, velocities and accelerations at the beginning of the increment. The advantage of the explicit analysis is that it is easy to solve the complicated contact problems because achieving numerical convergence is not needed. However, it is usually not used in quasi-static structural response problems (Yu 2008) due to some special considerations such as the step time. Since a quasi-static event is a long-time process, it is often computationally impractical to have the simulation in its natural time scale, which would require an excessive number of small time increments, even though it is not as accurate as the static solution of ABAQUS/Standard (Sarraj et al. 2007). Although the static method has difficulty in dealing with numerical non-convergence, it was decided to use this method in this research for the following reasons: firstly, many researchers have used it in numerical simulation of the behaviour of steel connection in fire with good accuracy; secondly, manipulation is easy. As will be explained later, pseudo-dynamic analysis, in the form of damping, will be introduced to deal with numerical non-convergence.

Therefore, in this research, the general-purpose finite element implicit solver, ABAQUS/Standard, was employed to conduct 3-D nonlinear FE simulations. Both material and geometric non-linearities were considered. In order to capture the large deformation and local instability effects in the 3-D FE models, the geometric nonlinearity was taken into consideration by activating the optimal parameter “NLGEOM” in ABAQUS/Standard.

In ABAQUS modeling of fire tests, the following six factors are highly influential on the simulation results and very important in determining a suitable modeling method:
material properties, element type and mesh configuration, contact properties, temperature, boundary condition and the stabilization factor. The following sections will describe the assumptions made in this research.

3.3.1 Materials properties

Material properties and stress states can be affected by temperature. In this research, the high temperature mechanical properties recommended in EN 1993-1-2 were adopted. The Eurocode material models are represented below.

3.3.1.1Concrete

The strength and deformation properties of uniaxially stressed concrete at elevated temperatures are presented in Figure 3.7. The stress-strain relationships given in Figure 3.7 are defined by two parameters; The compressive strength $f_{c,0}$ and the strain $\varepsilon_{c1,0}$ corresponding to $f_{c,0}$. Values for each of these parameters are given in Table 3.2 as a function of concrete temperature. For intermediate values of the temperature, linear interpolation can be used. The parameters specified in Table 3.2 may be used for normal weight concrete with siliceous or calcareous (containing at least 80% calcareous aggregate by weight) aggregates. Values for $\varepsilon_{cu1,0}$ defining the range of the descending branch may be taken from Table 3.2, where the values in column 4 are for normal weight concrete with siliceous aggregates, those in column 7 are for normal weight concrete with calcareous aggregates.

3.3.1.2 Structural steel

The strength and deformation properties of steel at elevated temperatures are specified in Figure 3.8 and Table 3.3, from EN 1993-1-2. The stress-strain relationships given in Figure 3.8 are defined by three parameters; the slope of the linear elastic range $E_{s,0}$, the proportional limit $f_{sp,0}$ and the maximum stress level $f_{sy,0}$. Also Figure 3.9 represents the reduction factors for strength and elastic modulus of carbon steel at elevated temperatures (EN 1993-1-2). For intermediate values of the temperature, linear interpolation can be used.
CHAPTER 3– NUMERICAL MODELLING OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMNS IN FIRE

Figure 3.7: Mathematical model for stress-strain relationships of concrete under compression at elevated temperatures (EN 1992-1-2, CEN 2004)

Table 3.2: Values for the main parameters of the stress-strain relationships of normal weight concrete with siliceous or calcareous aggregates concrete at elevated temperatures (EN 1992-1-2, CEN 2004)

<table>
<thead>
<tr>
<th>Concrete temp $\theta$ [°C]</th>
<th>Siliceous aggregates</th>
<th></th>
<th></th>
<th></th>
<th>Calcareous aggregates</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{c,e}/f_{c,e}$</td>
<td>$\varepsilon_{1,e}$</td>
<td>$\varepsilon_{\text{qu},e}$</td>
<td>$f_{c,e}/f_{c,e}$</td>
<td>$\varepsilon_{1,e}$</td>
<td>$\varepsilon_{\text{qu},e}$</td>
<td>$f_{c,e}/f_{c,e}$</td>
<td>$\varepsilon_{1,e}$</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>1,00</td>
<td>0,0025</td>
<td>0,0200</td>
<td>1,00</td>
<td>0,0025</td>
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<td>0,0250</td>
<td>0,97</td>
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<td>0,0250</td>
<td></td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>0,85</td>
<td>0,0070</td>
<td>0,0275</td>
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<td>0,0275</td>
<td></td>
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</tr>
<tr>
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<td></td>
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<tr>
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<td>0,0250</td>
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<tr>
<td>700</td>
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<td>0,0375</td>
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<tr>
<td>800</td>
<td>0,15</td>
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<td>0,0400</td>
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<td></td>
</tr>
<tr>
<td>900</td>
<td>0,08</td>
<td>0,0250</td>
<td>0,0425</td>
<td>0,15</td>
<td>0,0250</td>
<td>0,0425</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1000</td>
<td>0,04</td>
<td>0,0250</td>
<td>0,0450</td>
<td>0,06</td>
<td>0,0250</td>
<td>0,0450</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1100</td>
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<td>-</td>
<td>-</td>
<td>0,00</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
CHAPTER 3– NUMERICAL MODELLING OF RESTRAINED STRUCTURAL SUBASSEMBLIES
OF STEEL BEAM CONNECTED TO CFT COLUMNS IN FIRE

Figure 3.8: Mathematical model for stress-strain relationships of steel at elevated temperatures (EN 1993-1-2, CEN 2005)

Figure 3.9: Reduction factors for strength and elastic modulus of carbon steel at elevated temperatures (EN 1993-1-2, CEN 2005)
Table 3.3: Values for the main parameters of the stress-strain relationships of steel at elevated temperatures (EN 1993-1-2 CEN 2005)

| Steel temperature $\theta_a$ (°C) | Reduction factors at temperature $\theta_a$ relative to the value of $f_Y$ or $E_a$ at 20°C (subscript ‘a’ denotes ‘ambient temperature’)
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Effective Yield Strength (relative to $f_Y$) $k_{Y,\theta} = f_{Y,\theta} / f_Y$</td>
</tr>
<tr>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>20</td>
<td>1</td>
</tr>
<tr>
<td>100</td>
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<tr>
<td>200</td>
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<td>500</td>
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<td>1100</td>
<td>0.02</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
</tr>
</tbody>
</table>

The stress-strain constitutive relationships adopted in the FE models for the steel beams, columns and connection components were based on the steel tensile coupon tests at ambient temperature (Ding and Wang 2007) and the elevated temperature model in EN 1993-1-2. For ABAQUS simulation, the nominal engineering stress-strain model
obtained from steel tensile coupon test was converted to the true stress-strain relationship according to;

\[ \sigma_{true} = \sigma_{nom} (1 + \varepsilon_{nom}) \]

and

\[ \varepsilon_{true} = \ln (1 + \varepsilon_{nom}) \]

Where:
- \( \sigma_{true} \) and \( \varepsilon_{true} \) represent the true stress and strain
- \( \sigma_{nom} \) and \( \varepsilon_{nom} \) are the nominal stress and strain respectively.

Since no mechanical testing was performed on the bolts and nuts, their mechanical properties were assumed to be elastic-perfectly-plastic. For Grade 8.8 bolts, the yield strength was 640MPa and the elastic modulus was 210000MPa. The welds between connection components were simulated using the "tie" type constraint in ABAQUS.

### 3.3.2 Element Type, Mesh Size and Modelling Geometry

The type of element was investigated by many researchers. Kukreti et al. (1976) analyzed thirteen bench-mark connections by using 2-D shell elements and 3-D brick elements, so that their correlation characteristics could be applied for prediction of other 3-D values from corresponding 2-D results. Krishnamurthy (1979) developed two-dimensional and three-dimensional models of three different types of connections, top angle, end plate and tee stub connections. Patel and Chen (1985) concluded that the 2-D simplification may cause some deviations from the true three-dimensional response. Norbert et al. (1994) conducted a comparative analysis between the two-dimensional model and the three dimensional model and it was clearly shown that the quality of the results depended a lot on the finite element model chosen. From the comparison made between 2-D and 3-D models with the experimental results, Norbert concluded that the 2-D model was too stiff for the presentation of the real deformation and the three-dimensional finite element model could generally be recommended while the two-dimensional model was advisable only for special investigations. Al-Khatab and Bouchair (2007) analyzed T-stubs using a finite element model. The 3D model in the
finite elements software Cast3m took into account elastic plastic behaviour of the materials, large displacement and the unilateral contact between the connected parts. The 2-D models are certainly easier to implement. In order to define the limits of these models, a comparison was made between 2-D and 3-D models. The 2-D model gave satisfactory results for short T-stubs. For some geometries, only the 3-D model with consideration of contact, material and geometrical non-linearities allowed a faithful representation of the T-stub behaviour.

As can be seen from the previous short review, using three-dimensional finite elements is the best choice for investigation of joint behaviour in details. The ABAQUS element library offers a number of hexahedron, shell, contact and beam elements with different features. For nonlinear problems, the first order elements are likely to be the best option because of their good accuracy and low demand on computation resources. In this study, Three-dimensional solid elements (C3D8R) were used to model the main structural members. The advantage of this type of element is that it is accurate in modeling constitutive law integration and can remedy the shear lock problem when used with very dense mesh finite element model. To obtain accurate results, fine mesh was applied in the connection region where high stress and strain gradient would take place.

The tested structure was symmetrical in geometry. Therefore, to save computational time, it was decided to include only half of the test assembly in the finite element model. Figure 3.10 shows an example of the finite element model for a reverse channel connection test (Test4). Furthermore, to reduce the number of elements and nodes in the FE model, the column was divided into three parts and only the central part connected by the joint and exposed to fire in the furnace was actually modelled using solid elements. The other two parts away from the joint zone were modelled using general beam elements with “box” cross section for the steel tube and “rectangular” cross section for the concrete infill. The ABAQUS “Coupling” function was used to join the three column parts as shown in Figure 3.11. Since the column was not loaded, it was not necessary to refine the concrete infill temperature model.
Figure 3.10: 3-D FE model of reverse channel connections, Test 4 of Ding and Wang (2007)
3.3.3 Contact

Many engineering problems involve contact between two or more components. In these problems a force normal to the contacting surfaces acts on the two bodies when they touch each other. If there is friction between the surfaces, shear forces may be created that resist the tangential motion (sliding) of the bodies. The general aim of contact simulations is to identify the areas on the surfaces that are in contact and to calculate the contact pressures generated. In a finite element analysis, contact conditions are a special class of discontinuous constraint, allowing forces to be transmitted from one part of the model to another. The constraint is discontinuous because it is applied only when the two surfaces are in contact. When the two surfaces separate, no constraint is applied. The analysis has to be able to detect when two surfaces are in contact and apply the contact constraints accordingly. Similarly, the analysis must be able to detect when two surfaces separate and remove the contact constraints. By default, Abaqus/Standard uses a pure master-slave contact algorithm: nodes on one surface (the slave) cannot penetrate...
the segments that make up the other surface (the master). A consequence of this strict master-slave formulation is that the slave and master surfaces must be carefully selected to achieve the best possible contact simulation. The following simple rules must be followed: the slave surface should be the more finely meshed surface; and if the mesh densities are similar, the slave surface should be the surface with the softer underlying material (AB AQUS analysis user’s manual version 6.6, 2006).

In the test structures to be modelled in this research, many contact pairs exist in the joints, such as end plate to column, fin plate to beam web, bolt shanks and nuts to the connected members. The ABAQUS contact function was used to simulate the interaction between the contact pairs. A contact was defined as surface to surface contact with a small sliding option. “Hard contact” was assumed for the normal contact behaviour and a friction coefficient of 0.3 was used in the tangential direction of the contact pairs. The friction coefficient values have little effect on the simulation results Dai et al. (2010).

### 3.3.4 Boundary condition & temperature

The boundary conditions of the FE model were according to those in the test. The bottom of the columns was pinned in all three directions and the top of the columns was pinned in two directions but movement along the column axis was allowed. Since only half of the beam was included in the FE model due to symmetry as discussed earlier, the beam mid-section was fixed in the axial direction, which effectively prevented rotation about the two principal axes of the beam cross-section, but allowed the beam to twist about its longitudinal axis. As in the test, the FE modelling applied the loads in two steps: (i) two point loads were applied to the beam at ambient temperature; (ii) while maintaining the structural loads, the structural temperatures were increased until the end of the fire test.

In the FE model, six different temperature curves based on test measurements were adopted for different parts of the structure: three different temperatures for the beam (bottom flange, web and top flange); one temperature curve for the joint zone (all the
bolts, nuts and connection components such as fin plate, T-stub, reverse channel and endplate) as well as 100 mm length of the beam in the joint zone; one temperature curve for the steel tubular column in the joint region; one temperature curve for the concrete fill in the joint region. The temperature of the column away from the joint zone was set at ambient temperature.

### 3.3.5 Stabilization

Simulation of the restrained structural subassemblies of steel beam to CFT column in fire is a highly nonlinear problem. Therefore, the model can be temporarily unstable for reasons of a geometrical nature, such as buckling or a material nature, such as material softening. ABAQUS/Static solver offers an automatic mechanism in order to stabilize such unstable problem by applying volume-proportional damping to the model.

In static response, the basic statement of equilibrium is that, the internal forces, I, must balance the external forces, F,

\[ F - I = 0 \]  \hspace{1cm} -(3.3)

When the automatic stabilization is included in the nonlinear quasi-static procedure, the equilibrium equations takes this form;

\[ F - I - F_v = 0 \]  \hspace{1cm} -(3.4)

Where \( F_v = c M^* \nu \) is the viscous forces

In which:

- \( M^* \) is an artificial mass matrix calculated with unit density,
- \( c \) is a damping factor,
- \( \nu = \Delta u/\Delta t \) is the vector of nodal velocities,
- \( \Delta t \) is the increment of time (which may or may not have physical meaning in the context of the problem being solved).

While the model is stable the artificial damping has no effect. When the unstable condition is developed in the model, the local velocities increase and, consequently, part of the strain energy released is dissipated by the applied damping. Abaqus/Standard
can, if necessary, reduce the time increment to permit the process to occur without the unstable response causing very large displacements (ABAQUS analysis user’s manual version 6.6, 2006).

Since in fire condition, there were always unstable numerical simulations, the stabilization method should be used in the model to overcome the numerical non-convergence problem. The automatic stabilization scheme with a constant damping factor typically works well to tackle instabilities without having a major effect on the solution. However, there is no guarantee that the value of the damping factor is optimal or even suitable in some cases. In some cases the damping factor may have to be increased if the convergence behaviour is problematic or it may have to be decreased if it affects the solution. The former case would be required to rerun the analysis with a larger damping factor, while the latter case would be required to perform post-analysis comparison of the energy dissipated by viscous damping (ALLSD) to the total strain energy (ALLIE), and comparing the viscous forces (VF) with the overall forces in the analysis. Therefore, obtaining an optimal value for the damping factor is a manual process requiring trial and error until a converged solution is obtained while the dissipated stabilization energy is sufficiently small and the viscous forces should be relatively small compared with the overall forces in the model. Unfortunately, the damping factor is model dependent; therefore, it relies on the experience from the previous runs (ABAQUS analysis user’s manual version 6.6, 2006).

3.4 Sensitivity study results

Simulating the fire tests was a considerable undertaking and a lot of assumptions had to be made. Therefore, it is important to check how the simulation results may change when the assumed value of a parameter changes. The sensitivity study may then be used to further validate the numerical simulation model and to also give some indication on the level of refinement necessary to achieve acceptable results. In this section, the sensitivity of numerical simulation results to mesh size, steel grade, gap size between the bolt and the plate and lateral and torsional restraint at the four corners of the load
plate (see Figure 3.10b) will be investigated. In addition, this section will also explain how an appropriate stabilization factor may be determined.

3.4.1 Mesh sensitivity

A series of ABAQUS models were built and run to assess the sensitivity of simulation results to the FE mesh. Similar to the results of Dai et al. (2010), the results of this sensitivity study suggest that the appropriate mesh size would be 10-20mm for the main structural members such as the beam, the joint components and the column. Too small an element size would consume too much computer time, while a too coarse mesh (Figure 3.12b) would not be able to reveal some important member buckling characteristics such as shown in Figure 3.12b. Also, in order to avoid premature beam web buckling, at least two layers of elements in the thickness direction of the beam web should be used. Figure 3.13 compares the simulated and measured beam axial force and mid-span deflection curves for reverse channel connection (Test 4) for the two cases. It can be seen that the case of suitable mesh gives better agreement with the test results.

(a) Suitable beam mesh

(b) Coarse beam mesh

<table>
<thead>
<tr>
<th>Number of nodes</th>
<th>29063 (a)</th>
<th>10702 (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of elements</td>
<td>17799 (a)</td>
<td>5715 (b)</td>
</tr>
</tbody>
</table>
CHAPTER 3– NUMERICAL MODELLING OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMNS IN FIRE

Figure 3.12: Effect of element sizes on structural deformation of Test 4

(a) The axial forces
CHAPTER 3– NUMERICAL MODELLING OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMNS IN FIRE

3.4.2 Mechanical property sensitivity

In the numerical modelling, the average yield strength, ultimate strength and elastic modulus of the steel members at ambient temperature were used based on the steel coupon tests. Due to missing precise tensile coupon test data for both the end plate and the reverse channel in Test 4 and Test 5, simulations were done for ambient temperature yield stress values of 275 N/mm² and 350 N/mm² and the most similar results to the test were taken. As shown in Figure 3.14 & 3.15, using 275 N/mm² as ambient temperature yield stress for the endplate and reverse channel gave better agreement with the test results for the deformation shape. Nevertheless, because failure was caused by the beam, this lack of precise information on the reverse channel did not affect the simulation results during the transition from beam compression to catenary action, as shown in Figure 3.16.
Figure 3.14: Deformed shapes of FE model using different ambient temperature yield stresses for reverse channel and endplate in Test 4 of Ding & Wang (2007)

Figure 3.15: Deformed shapes of FE model using different ambient temperature yield stresses for reverse channel and endplate in Test 5 of Ding & Wang (2007)

(a) The axial forces
3.4.3 Contact parameters

In the test structures, many contact pairs existed in the joints. For example, these contact pairs occurred at the interfaces of the following contact points for reverse channel connection: the bolt heads and web of reverse channel; the bolt nuts and endplate; the bolt shanks and web of reverse channel; the bolt shanks and endplate; and the web of reverse channel and endplate. The ABAQUS contact function was used to simulate the interaction between the contact pairs. In order to reduce the computational cost, all contact relationships were defined as “surface to surface” contacts with a small sliding option. “Hard contact” was assumed for the normal contact behaviour and a friction coefficient of 0.3 was used in the tangential direction of the contact pairs. The friction coefficient values have been found to have little effect on the simulation results (Dai et al. 2010). The clearance between the bolt shanks and bolt holes was assumed 1 mm in all numerical simulations.
The effect of gap size between the bolt and the plate in the simulation results for all tests is investigated. For example, Figure 3.17 compares the simulated and measured beam axial force and mid-span deflection curves for bolted T-stub connection (Test 2) using no gap and 1 mm gap size between the bolt and the plate. It can be seen that throughout thermal expansion stage, the compression forces significantly decrease with the increase of the gap size between the bolt and the plate. This was caused by the contact area between the bolt and the plate in case of 1 mm gap size being lower than in case of no gap. The beam mid span deflection was slightly affected by the gap size between the bolt and the plate.
3.4.4 Lateral torsional restraint

As mentioned in section 3.2.2, there was no deliberate application of lateral or torsional restraint to the beam. However, in the test, in order to ensure that the loading jacks remained attached to the beam, each loading jack was inserted into a steel bracket which was then clamped on the top flange of the beam, as shown in Figure 3.2. Simulations were carried out to assess the influence of this loading system on structural behaviour. Figure 3.18 compares the simulated overall deformed shapes of the same structure with or without lateral restraints at the four corners of the load plate, for Test 4. It can be seen that in case (a) where the lower flange was prevented from moving laterally and twisting, the simulation results give better agreement with the test result than case (b) where both flanges of the beam moved laterally and twisted. Figure 3.19 further compares the simulated and measured beam axial force and mid-span deflection curves for Test 4 for the two cases. It can be seen that the case of lateral restraints at the four corners of the load plate again gives better agreement with the test results. This
behaviour has been observed in the simulations of all connection types, as shown in Figures 3.20-3.22.

(a) Lateral restraints

(b) No lateral restraints

(c) Test - Ding and Wang (2007)

Figure 3.18: Effect of lateral restraint at the four corners of the load plate in Test 4 of Ding & Wang (2007)
Figure 3.19: Comparison between FE (with and without lateral restraint at the corners of the loading plate) and Test 4 of Ding & Wang (2007) results for mid-span deflections and axial forces
CHAPTER 3– NUMERICAL MODELLING OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMNS IN FIRE

Figure 3.20: Effect of lateral restraint at the corners of the load plate in Test 3 of Ding & Wang (2007)

Figure 3.21: Effect of lateral restraint at the corners of the load plate in Test 6 of Ding & Wang (2007)
3.4.5 Restraint at the column ends

In the numerical model, the column ends were assumed to be able to rotate freely in the plane of the structure. However, as indicated in Figure 3.1, the column flange at the ends was linked to the load cell at two locations using bolts. This arrangement may induce some rotational restraint at the column ends. The effect of this additional restraint at the column ends should be assessed to ensure that the boundary conditions in the numerical simulation model was correct. Additional models of the structure for tests 5, 8 and 10 were created assuming complete rotational fixity at the column ends. Figure 3.23 compares the simulation results for the models with pin and rigid column ends for these tests. With complete fixity at the column ends, the axial force in the beam, both in compression and in tension, is much greater than the measured results. In contrast, assuming pinned column ends produced results much closer to the test results.
It can therefore be accepted that the actual column ends would be much closer to pin support than fixed end support, and this will be assumed in all the numerical simulation models.

![Graph](a) Test 5

![Graph](b) Test 8
3.4.6 Artificial damping – energy dissipation factor

Simulation of the restrained structural subassemblies studied in this research represents a highly nonlinear problem. Therefore, the numerical model can become unstable, caused by problems of a geometrical nature, such as buckling or of a material nature, such as material softening. For ambient temperature analysis, it would be possible to apply the Riks method but for simulations with changing temperatures, the Riks method would not be suitable. Instead, this research applied an artificial volume-proportional damping to the model. In general, if the damping factor is too low, it would not be able to prevent temporary structural failure from terminating the simulation. On the other hand, if the damping factor is too high, the artificial damping may absorb a large fraction of the total energy applied and the simulated structural behaviour may not be representative of the real behaviour of the structure. In this study, a series of ABAQUS models, using different damping factors, for each test were built and run to select a proper dissipated energy fraction in order to tackle the non-convergence problem while producing the least effect on the structural behaviour. To check whether a particular
damping factor is appropriate, the simulated reaction forces can be compared with the applied loads. In principle, the reaction forces should be in static equilibrium with the applied loads. However, if the structure experiences instability, the artificial damping may contribute to resisting the applied load, causing the reaction forces to be lower than the applied loads. If the instability is temporary, the damping effect should decrease and the reaction forces should return to the level of the applied loads. However, if the instability is genuinely caused by structural failure, the artificial damping will continue to be effective and the reaction forces will continue to decrease.

Figure 3.24 compares the applied load (30kN) with the vertical reaction force using different damping factors. The maximum average temperature reached in the bottom flange for Test 4 was 733°C. When a damping factor of 0.000005 was used, the numerical model experienced non-convergence at a beam temperature slightly below the measured maximum beam temperature. In order to check whether this was caused by real structural failure or numerical instability, numerical models with higher values of damping factor were run. Figure 3.25 compares the simulation results of these different numerical models. Compared to the model using a damping factor of 0.000005, using a damping factor of 0.00001 reached a slightly higher beam temperature without loss of the applied load before the reaction force dropping significantly below the applied load. Models with higher damping factors continued to run past the measured maximum beam temperature, however, at temperatures higher than the measured maximum beam temperature, the reaction force in the structure continued to be much lower than the applied load. This clearly indicates that for these models using high damping factors, numerical stability could only be achieved by reducing the applied load. In other words, the sharp drop in the reaction force from the models with damping factor of 0.000005 and 0.00001 is a genuine indication of structural failure.
Figure 3.24: Comparison of the vertical reactions from FE models using different dissipated energy fractions
To ensure consistency of the FE model, all the ten tests were simulated using the same parameters as determined from the above sensitivity study.

### 3.5 CONCLUSIONS

This chapter presents the 3-D finite element numerical simulation model using ABAQUS/Static to simulate the 10 fire tests recently conducted in the University of Manchester on restrained steel subassemblies using four different types of beam to column joint. The models incorporate nonlinear material properties for all the subassembly components, geometric non-linearity and contact interaction. Chapter 4 will present the validation results through comparison with the test results. From the results of the sensitivity studies presented in this chapter, the following conclusions may be drawn:

1. Two main assumptions used in this research; (a) uniform temperature distribution along the beam length based on the test results; (b) the structures are
symmetrical in geometry and boundary conditions before and after fire actions.

2. If an appropriate damping factor is used, the ABAQUS/Static solver has the ability to model very large structural deflections and severe component distortions at high temperatures. This chapter has proposed a method, as explained below, of how an appropriate damping factor may be chosen.

3. To overcome numerical non-convergence due to temporary instability in the structure, a pseudo damping analysis may be used by selecting an appropriate dissipated energy fraction (damping factor). To check that the damping factor used is appropriate, the reaction forces in the structure may be examined. This analysis enables the numerical procedure to converge. When the structure experiences failure, some of the applied loads would be resisted by damping so the reaction forces would be lower than the applied loads. If the aforementioned failure is permanent, the reaction forces will continue to be lower than the applied loads and will not be able to recover even though the numerical model produces converged solution. This should be discarded. However, if the structure only experiences local/temporary failure, the numerical solution will converge to the correct solution after the structure has recovered from the local/temporary failure. This can be discerned by observing that the reaction forces will recover to be in equilibrium with the applied loads.

4. A refined mesh is necessary to reveal detailed local deformation in the structure, such as web and flange local buckling. However, global behavioural results such as beam axial force and deflection are not seriously affected when the beam is in catenary action stage.

5. The gap size between the bolt and the plate has some effects on the axial compression forces throughout thermal expansion stage, mainly in the fin plate and bolted T-stub connections. However, the effect is minimal in the important stage of large deflection behaviour of the structure. This is not surprising because by then any gap effect will be overwhelmed by the large deflection behaviour of the structure.
6. For the specific test arrangement adopted in Ding and Wang (2007), the loading jacks provided the beam with lateral and torsional restraints, preventing the beam from the laterally displacement and twisting.

7. Also, the assumption of assuming pinned column ends gives much closer agreement between the FE results and test results in terms of the axial force in the beam, both in compression and in tension, compared to assuming fixed column end supports. Therefore the pin support will be used in all the numerical simulation models.
CHAPTER 4 – VALIDATION OF THE NUMERICAL SIMULATION MODEL

4.1 INTRODUCTION

In the previous chapter, the finite element software package, ABAQUS, was used to establish a numerical simulation model. This chapter will assess accuracy of the model by comparing the simulation results with the experimental results of Ding and Wang (2007).

4.2 VALIDATION OF ABAQUS SIMULATIONS

Two sets of independent results were used for this validation study. The first set is for the benchmark exercise proposed by Gillie and the second set is the ten different fire tests recently conducted at the University of Manchester.

4.2.1 Comparison with benchmarks of Gillie (2009)

Gillie (2009) demonstrates the complexity of heating induced effects on structures in a simple example structure, in which material non-linearity, geometric non-linearity, time varying forces, thermal expansion and restraints interact to give a variety of structural phenomena. As partial validation of the numerical modelling, the author has repeated this benchmark exercise. The example structure consists of a single beam, uniformly heated from 0 to 800°C and then cooled down again. Figure 4.1 gives the basic data for the beam, including dimensions, boundary conditions, loading, and variation of stress-strain relationships at elevated temperatures. This load was chosen so that it could just be sustained when the beam is simply supported and heated to 800°C. The beam material properties were assumed throughout heating and cooling phases to be representative of an elasto-plastic steel with a yield strength reducing linearly from 250MPa at 0°C to 0 at 1000°C, as shown in Figure 4.1. Although this arrangement does not allow the capability of simulating catenary action to be tested, it gives an opportunity to verify the basic simulation methodology. Since the beam is axially restrained, the behaviour of the beam agrees with that outlined by Wang (2002) and later numerically confirmed by Yin and Wang (2004&2005). Of particular importance
is the level of residual tensile force in the beam after cooling down, being greater than 50% of full tensile strength of the beam, even when only limited axial restraint is present.

Figure 4.1: Example problem definition (Gillie 2009)

Figure 4.2 and 4.3 compare the author’s simulation results with those of Gillie (2009). It is clear that these two sets of results agree with each other very well in terms of trends. In particular, the author’s model accurately simulated all the important phenomena of the structure, including: rapid increase in compression force followed by axial buckling which results in rapid increase in beam lateral deflection and rapid decrease in axial compression force; generation of significant amount of axial tension during cooling. There is some mismatch of results. For example, the results of Gillie’s benchmark for 75% support stiffness match with the predicted results for 100% stiffness. And the results of 50% stiffness benchmark case match with the predicted results of 75% stiffness and so on. This was believed to be a presentational error in Gillie’s paper. Using different step sizes also led to some difference in results, particularly the peak values. The large step size used by Gillie was not able to accurately simulate the peak values.
Figure 4.2: Axial force and deflections predicted by Gillie (2009)
Figure 4.3: Axial force and deflections predicted by the author
4.2.2 Comparison between finite element simulations and fire tests of Ding & Wang (2007)

4.2.2.1 Test 1: fin plate connection

Test 1 used SHS 200×5mm tubes and fin plate connection. The geometrical details of Test 1 are shown in Figure 4.4. Referring to this figure, 8mm thickness fin plate was welded to the middle of the tubular wall with FW 8mm on both sides and bolted to the beam with two M20 Grade 8.8 bolts. There was no fire protection on the joints.

Figure 4.5 shows the FE mesh. Figure 4.6 presents the measured temperature-time relationships for the beam web and flanges, the steel tube, the connection zone and the concrete fill. In the numerical modeling, the average yield strength, ultimate strength and elastic modulus of the steel members at ambient temperature were used and are given in Table 4.1, based on the steel coupon tests. For Test 1 the average concrete cube strength was 40.7 MPa and the density was 2247 kg/m³.
CHAPTER 4 – VALIDATION OF THE NUMERICAL SIMULATION MODEL

Figure 4.5: FE mesh for Test 1

![Figure 4.5: FE mesh for Test 1](image)

Figure 4.6: Time-temperature relationships used in Test 1

![Figure 4.6: Time-temperature relationships used in Test 1](image)

Table 4.1: Mechanical property values for different steel members in Test 1

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>Fin plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>220848</td>
<td>215354</td>
<td>197016</td>
<td>190386</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>353.5</td>
<td>328.8</td>
<td>341.7</td>
<td>310.6</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>470.2</td>
<td>470.1</td>
<td>455.8</td>
<td>468.1</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>27.3</td>
<td>31.2</td>
<td>27.4</td>
<td>37.2</td>
</tr>
</tbody>
</table>

Figure 4.7 compares the modeling and experimental results for deformation modes of the beam and the joint. It can be seen that the observed deformations of the joint
components and beam were followed by the numerical model. As shown in Figure 4.7(a), both the test and the numerical model show that the beam experienced large twist and vertical deflections. Figure 4.8 compares the measured and simulated beam axial force and beam mid-span deflection as functions of the beam lower flange temperature at mid-span. It shows very good agreement for the beam mid-span deflection but the finite element model overestimates the beam axial force during the thermal expansion stage. Nevertheless, the simulation results have accurately captured the transition process from compression to catenary action in the beam and the predicted beam axial force in catenary action stage matches the measured results very well. The test specimen failed by fracture of the weld at the top. Since weld was not directly modelled, this failure mode could not be simulated. Instead, the numerical model failure was in the fin plate close to the weld under tension and shearing.
(b) Deformed endplate

(c) Model plastic strain

Figure 4.7: Behaviour and failure mode of Test 1

(a) Beam axial forces – temperature relationships
CHAPTER 4 – VALIDATION OF THE NUMERICAL SIMULATION MODEL

4.2.2.2 Test 2: (bolted T-stub) connection

Test 2 used SHS 200×5mm tubes and bolted T-stub connection. Referring to Figure 4.9 a T-stub 133×102×13 was bolted to the tubular wall using four (2 by 2) M16 Molabolts (www.molabolts.co.uk) and bolted to the beam with two M20 Grade 8.8 bolts. There was no fire protection on the joints.

The FE mesh is shown in Figure 4.10. Figure 4.11 presents temperature-time relationships for the beam web and flanges, the steel tube, the connection zone and the concrete fill. Table 4.2 lists the average yield strength, ultimate strength and elastic modulus of the steel members at ambient temperature, based on the steel coupon tests. For Test 2 the average concrete cube strength was 40.7 MPa and the density was 2247 kg/m³.
CHAPTER 4 – VALIDATION OF THE NUMERICAL SIMULATION MODEL

Figure 4.9: Geometrical details of bolted T-stub connection in Test 2 of Ding & Wang (2007)

Figure 4.10: FE mesh for Test 2
CHAPTER 4 – VALIDATION OF THE NUMERICAL SIMULATION MODEL

![Graph showing time-temperature relationships used in Test 2](image)

**Figure 4.11: Time-temperature relationships used in Test 2**

**Table 4.2: Mechanical property values for different steel members in Test 2**

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>T-stub Web</th>
<th>T-stub Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>210051</td>
<td>226691</td>
<td>197016</td>
<td>186069</td>
<td>214146</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>395.5</td>
<td>378.6</td>
<td>341.7</td>
<td>427.1</td>
<td>426.9</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>549.5</td>
<td>572.3</td>
<td>455.8</td>
<td>551.8</td>
<td>566.9</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>25.3</td>
<td>27.2</td>
<td>27.4</td>
<td>26.4</td>
<td>24.1</td>
</tr>
</tbody>
</table>

Test 2 failed due to pull-out of the beam web at the left end, as clearly shown in Figure 4.12. This was predicted by the numerical model, as shown by the plastic strain patterns in Figure 4.12 (c). Parts of the T-sections bolted to the steel tubes were pulled out slightly. The steel tube walls were also pulled out slightly around the joint area but there was no indication of failure in the tubes, see Figure 4.12(a) & (b). This was accurately captured by the simulation results. There was no noticeable damage in the Molabolts. Therefore, the large rotation of the joint as shown in Figure 4.12(b) is mainly a result of bearing deformations of the web of the steel beam and T-stubs around the bolt holes. The simulated bolted T-stub deformation pattern is also very similar to the test observation. Figure 4.13 compares the simulated and measured beam axial force and mid-span deflection. The agreement is satisfactory overall.
Nevertheless, at the small beam deflection stage, the numerical results for beam deflection are considerably higher than the experimental results. This difference in the beam mid-span deflection may be due to the difficulty in experimentally recording the beam deflection when it was small. The beam deflection was measured by attaching the LVDT to a ceramic rod through the furnace lid and the ceramic rod may not have been able to promptly follow the movement of the beam due to friction between the rod and the furnace insulation lining. To substantiate this claim, the beam’s thermal bowing deflection was calculated and the results are shown in Figure 4.13(b). Since the beam’s deflection before experiencing accelerated rate of change was mainly thermal bowing deflection, the close agreement between the numerical simulation results and the analytical calculation results indicates that the numerical results are credible. The beam’s thermal bowing deflection at mid-span was calculated using the following equation:

\[ \delta_{th} = \frac{\alpha \Delta T L^2}{h} \]

Where;
\( \alpha \) is the thermal expansion coefficient of steel (0.000014 m/m.C)
\( \Delta T \) the temperature difference between the top and bottom flanges
L the beam span (2m) and \( h \) the overall beam depth (178mm)

The same explanation as above can be applied to the other tests. However, since the focus of this research is on stage of structural behavior when the beam’s deflections are large, the difference in the small deflection stage has little effect.
CHAPTER 4 – VALIDATION OF THE NUMERICAL SIMULATION MODEL

Figure 4.12: Behaviour and failure mode of Test 2

(a) Overall deformed shape

(b) Deformed bolted T-stub

(c) Model plastic strain
CHAPTER 4 – VALIDATION OF THE NUMERICAL SIMULATION MODEL

Beam axial forces – temperature relationships

(b) Beam mid-span deflections – temperature relationships

Figure 4.13: Comparison of modeling and experimental results for mid-span deflection and axial force in the beam restrained by bolted T-stub connection (Test 2)
4.2.2.3 Test 3: (bolted T-stub) connection

Test 3 used SHS 200×12.5mm tubes and bolted T-stub connection. The only difference between the test 3 and test 2 configurations was in the thickness of the tube. Referring to Figure 4.14, a T-stub 133×102×13 was bolted to the tubular wall using four (2 by 2) M16 Molabolts and bolted to the beam with two M20 Grade 8.8 bolts. The original intention of tests 2 and 3 was to investigate the effects of tubular wall thickness on connection behaviour. However, after Test 3, it was determined that because the connection failure was caused by fracture of the beam web, instead of the tubular wall, a length of 100mm of the beam web was protected by 15mm thick ceramic fiber blanket at both end, as shown in Figure 4.14.

Figure 4.15 shows the FE model and Figure 4.16 presents the temperature-time relationships for the beam web and flanges, the steel tube, the connection zone and the concrete fill. In the numerical modelling, the average yield strength, ultimate strength and elastic modulus of the steel members at ambient temperature were used as in Test 3. For Test 3 the average concrete cube strength was 52.8 MPa and the density was 2334 kg/m³.

![Figure 4.14: Geometrical details of bolted T-stub connection in Test 3 of Ding & Wang (2007)]
As shown in Figure 4.17, the simulation results accurately reproduced the deformation pattern in all parts of bolted T-stub joint specimen. Failure of this sub-frame was due to fracture of the stem of the T-section on the left hand side. In the simulation results neither noticeable deformation could be seen in the steel tubes nor was there a sign of tube failure. As shown in Figure 4.17 (b), there was no noticeable deformation in T-sections bolted to the steel tubes and the Molabolts on the right hand side. Figure 4.18 compares the simulated and measured beam axial force and mid-span deflection. The
predicted results slightly overestimate the compression axial forces but in catenary action stage the results match very well.

(a) Overall deformed shape

(b) Deformed bolted T-stub

Figure 4.17: Behaviour and failure mode of Test 3
Figure 4.18: Comparison of modeling and experimental results for mid-span deflection and axial force in the beam restrained by bolted T-stub connection (Test 3)
4.2.2.4 Test 4: reverse channel connection

Test 4 used SHS 200×12.5mm tubes and reverse channel connection. Referring to Figure 4.19, the flanges of a reverse channel section 152×76 were welded to the tubular wall with FW 8mm on both sides. An endplate of 8mm thickness was welded to the beam web with FW 6mm on both sides and then bolted to the reverse channel using four (2 by 2) M20 Grade 8.8 bolts. There was no fire protection on the joints.

Figure 4.20 shows the FE mesh and Figure 4.21 presents the temperature-time relationships for the beam web and flanges, the steel tube, the connection zone and the concrete fill. In the numerical modelling, the average yield strength, ultimate strength and elastic modulus of the steel members at ambient temperature were used and are given in Table 4.3, based on the steel coupon tests. For test 4 the average concrete cube strength was 39.1 MPa and the density was 2293 kg/m³.

Figure 4.19: Geometrical details of reverse channel connection in Test 4 of Ding & Wang (2007)
Figure 4.20: FE mesh for Test 4

Figure 4.21: Time-temperature relationships used in Test 4

Table 4.3: Mechanical property values for different steel members in Test 4

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>End plate</th>
<th>Reverse channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>220848</td>
<td>215354</td>
<td>203213</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>353.5</td>
<td>328.8</td>
<td>491.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>470.2</td>
<td>470.1</td>
<td>535.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>27.3</td>
<td>31.2</td>
<td>19.4</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 4.22 compares the modelling and experimental results for deformation modes of the beam and the joint. It can be seen that the observed deformations of the joint
components and beam were followed by the numerical model. Figure 4.23 compares the measured and simulated beam axial force and beam mid-span deflection as functions of the beam lower flange temperature at mid-span. The test structure experienced larger compression during the thermal expansion stage, which was accompanied by the smaller beam mid-span deflection. The simulation results accurately captured the transition process from compression to tension in the beam. As shown in Figure 4.22 (a), failure of this sub-frame was due to fracture in the beam web on the left hand side which was predicted by the numerical model, as shown by the tensile stress patterns in Figure 4.22(d). The fracture length was the same as the weld length and the fractured web formed a sharp edge, as shown in Figure 4.22(c). Due to large deflections in the beam and the flexibility of the joint, the endplates and the front faces of the reverse channels deformed significantly, which was captured by the numerical model as shown in Figure 4.22(a)-(c). The side faces of the simulated reverse channels turned inside out at the top which is similar to the test observation, as shown in Figure 4.22(a)&(b), under combined tension and bending moment. In the numerical model there was neither fracture in the reverse channels nor any damage found in the connection welds, this is also similar to the test observation.

(a) Overall deformed shape
CHAPTER 4 – VALIDATION OF THE NUMERICAL SIMULATION MODEL

(b) Deformed endplate & reverse channel

(c) Deformed reverse channel

(d) Model tensile stress

Figure 4.22: Behaviour and failure mode of Test 4

(a) Beam axial forces – temperature relationships
127

(b) Beam mid-span deflections – temperature relationships

Figure 4.23: Comparison of modeling and experimental results for mid-span deflection and axial force in the beam restrained by reverse channel connection (Test 4)

4.2.2.5 Test 5: reverse channel connection

Test 5 used SHS 200×5mm tubes and reverse channel connection. The only difference between Test 5 and Test 4 configurations was the tube wall thickness. Referring to Figure 4.24, the flanges of a reverse channel section 152×76 was welded to the tubular wall with FW 8mm on both sides. An endplate of 8mm thickness was welded to the beam with FW 6mm on both sides and then bolted to the reverse channel with four (2 by 2) M20 Grade 8.8 bolts. Similar to tests 2 and 3, tests 4 and 5 were again intended to investigate the effects of tubular wall thickness. However, again, since the connection failed due to fracture of the beam web in Test 5, instead of the tubular wall, a length of 100mm of the beam web and the endplate were protected by 15mm ceramic fibre blanket at both ends as shown in Figure 4.24.

Figure 4.25 shows the FE mesh and Figure 4.26 presents the input temperature-time relationships for the beam web and flanges, the steel tube, the connection zone and the concrete fill. In the numerical modelling, the average yield strength, ultimate strength and elastic modulus of the steel members at ambient temperature were used and are
given in Table 4.4, based on the steel coupon tests. For Test 5 the average concrete cube strength was 44.4 MPa and the density was 2301 kg/m$^3$.

Table 4.4: Mechanical property values for different steel members in Test 5

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>End plate</th>
<th>Reverse channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>210051</td>
<td>226691</td>
<td>197016</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>395.5</td>
<td>378.6</td>
<td>341.7</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>549.5</td>
<td>572.3</td>
<td>455.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>25.3</td>
<td>27.2</td>
<td>27.4</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 4.24: Geometrical details of reverse channel connection in Test 5 of Ding & Wang (2007)

Figure 4.25: FE mesh for Test 5
Figure 4.26: Time-temperature relationships used in Test 5

Figure 4.27 compares the modelling and experimental results for deformation modes of the beam and the joint. It can be seen that the observed deformations of the joint components and beam were followed by the numerical model. Figure 4.28 compares the measured and simulated beam axial force and beam mid-span deflection as functions of the beam lower flange temperature at mid-span. Figure 4.28 also shows that the predicted beam axial force and beam mid-span deflection match the measured results very well. The test structure experienced larger compression during the thermal expansion stage, which was accompanied by the smaller beam mid-span deflection. The simulation results accurately captured the transition process from compression to tension in the beam. The test specimen failed by fracture of the weld between the beam and the endplate at the top. Since weld was not directly modelled, this failure mode could not be simulated. Instead, the numerical model failure was in the endplate close to the weld under tension, as shown in Figure 4.27(b) & (d). Due to the large catenary force in the beam, the endplates and the reverse channels distorted significantly, this is captured by the numerical model as shown in Figure 4.27(b)-(e).
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Figure 4.27: Behaviour and failure mode of Test 5

(a) Overall deformed mode
(b) Deformed end plate
(c) Deformed reverse channel
(d) Deformed shape at joint zone
(e) Model tensile stress

Figure 4.27: Behaviour and failure mode of Test 5
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(a) Beam axial forces – temperature relationships

(b) Beam mid-span deflections – temperature relationships

Figure 4.28: Comparison of modeling and experimental results for mid-span deflection and axial force in the beam restrained by reverse channel connection (Test 5)
4.2.2.6 Test 6: extended end plate connection

Test 6 used SHS 200×5mm tubes and extended endplate connection. Referring to Figure 4.29, an extended endplate of 8mm thickness was bolted to the tubular wall using six (3 by 2) M16 Molabols and was welded to the beam with FW 6mm around the beam section. There was no fire protection on the joints. Figure 4.30 shows the FE mesh and Figure 4.31 shows the input temperature-time relationship for the beam web and flanges, the steel tube, the connection zone and the concrete fill. In the numerical modelling, the average yield strength, ultimate strength and elastic modulus of the steel members at ambient temperature were used and are given in Table 4.5, based on the steel coupon tests. For Test 6 the average concrete cube strength was 51.0 MPa and the density was 2290 kg/m³.

Figure 4.29: Geometrical details of extended endplate connection in Test 6 from Ding & Wang (2007)
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Figure 4.30: FE mesh for Test 6

Figure 4.31: Time-temperature relationships used in Test 6

Table 4.5: Mechanical property values for different steel members in Test 6

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>Endplate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>206606</td>
<td>206378</td>
<td>200001</td>
<td>190386</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>351.5</td>
<td>309.3</td>
<td>406.2</td>
<td>310.6</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>493.2</td>
<td>477.0</td>
<td>539.4</td>
<td>468.1</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>32.1</td>
<td>34.9</td>
<td>29.5</td>
<td>37.2</td>
</tr>
</tbody>
</table>

Figure 4.32(a) & (b) compares the deformation patterns in various joint components and in the beam and the column. The simulation results agree with the test observation in
most cases. Failure was due to pull-out of the Molabolts. However, due to a lack of precise information on Molabolts, this failure mode could not be simulated. Nevertheless, significant pull-out of the bolts was observed in the numerical model. In addition, significant distortion of the endplates and pull-out of the steel tube walls around the joint area were predicted by the numerical model. The predicted beam axial force and mid-span deflection characteristics match satisfactory with the experimental observation as shown in Figure 4.33.

(a) Overall deformed shape

(b) Deformed endplate

Figure 4.32: Behaviour and failure mode of Test 6
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Figure 4.33: Comparison of modeling and experimental results for mid-span deflection and axial force in the beam restrained by extended endplate connection (Test 6)
4.2.2.7 Test 7: extended end plate connection

This test had the same arrangement as test 6. However, the applied load was only half that of test 6 (being equal to that in tests 1-5, i.e. 30 KN) with the intention to avoid the Molabolts failure mode observed in Test 6. The time temperature relationship for the beam web and flanges, the steel tube, the connection zone and the concrete fill are shown in Figure 4.34. For Test 7 the average concrete cube strength was 54.8 MPa and the density was 2313 kg/m³.

![Figure 4.34: Time-temperature relationships used in test 7](image)

Since the Molabolts were the same as in test 6, the failure mode of this test is the same as that in Test 6, except for the bottom two Molabolts on the right hand side column that were not damaged. Figure 4.35(a) & (b) compares the deformation patterns in various joint components and in the beam and the column. The simulation results agree with the test observations, including pull out of the Molabolts, significant distortion of the endplates and slight pull out of the steel tube walls around the joint area. The predicted beam axial force and mid-span deflection characteristics match the experimental observation well with a little increase in beam axial force and mid-span deflection as shown in Figure 4.36.
Figure 4.35: Behaviour and failure mode of Test 7

(a) Overall deformed shape

(b) Deformed endplate
Figure 4.36: Comparison of modeling and experimental results for mid-span deflection and axial force in the beam restrained by extended endplate connection (Test 7)
4.2.2.8 Test 8: reverse channel connection with flush endplate

Test 8 used CHS 193.7×5mm tubes and reverse channel connection. Referring to Figure 4.37 the flanges of a reverse channel section 152×89 were welded to the tubular wall with FW 8mm on the outside. A flush endplate of 8mm thickness was welded to the beam with FW 6mm around the beam section and then bolted to the reverse channel with four (2 by 2) M20 Grade 8.8 bolts. There was no fire protection on the joints. This test had an improved reverse channel connection compared to tests 4 & 5 by using a flush endplate welded to the steel beam. Also a CHS tube was used, instead of a SHS tube used in tests 4 & 5. The applied load in this test was 1.5 times that of tests 1-5 on the assumption that the joint would be able to develop about 50% of the beam bending moment capacity, so maintaining the same load ratio as in tests 1-5.

Figure 4.38 shows the FE model and Figure 4.39 presents the input temperature-time relationship for the beam web and flanges, the steel tube, the connection zone and the concrete fill. In the numerical modelling, the average yield strength, ultimate strength and elastic modulus of the steel members at ambient temperature were used and are given in Table 4.6, based on the steel coupon tests. For Test 8 the average concrete cube strength was 45.7 MPa and the density was 2207 kg/m³.
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Figure 4.38: FE mesh for Test 8

Figure 4.39: Time-temperature relationships used in Test 8

Table 4.6: Mechanical property values for different steel members in Test 8

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>End plate</th>
<th>Reverse channel web</th>
<th>Reverse channel flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>206606</td>
<td>206378</td>
<td>200001</td>
<td>-</td>
<td>192280</td>
<td>207811</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>351.5</td>
<td>309.3</td>
<td>406.2</td>
<td>-</td>
<td>396.8</td>
<td>351.4</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>493.2</td>
<td>477.0</td>
<td>539.4</td>
<td>-</td>
<td>562.2</td>
<td>560.7</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>32.1</td>
<td>34.9</td>
<td>29.5</td>
<td>-</td>
<td>27.1</td>
<td>32.6</td>
</tr>
</tbody>
</table>

Test 8 failed due to thread-stripping of the nuts on the left hand side and fracture of the front face of the right hand side reverse channel, as clearly shown in Figure 4.40(a).
While the thread-stripping of the nuts on the left hand side failure mode could not be reproduced by the simulation model because thread stripping was not included in the model, the fracture of the front face of the right hand side reverse channel is clearly predicted by the numerical model, as shown in Figure 4.40(b) & (c). The simulated flexible endplate and reverse channel deformation pattern is also very similar to the test observation, where the endplates and the front faces of the reverse channels also suffered from large deformation, as shown in Figure 4.40(c)-(e). There was no noticeable deflection in the steel tubes. Neither was any damage found in the connection welds. Figure 4.41 compares the simulated and measured beam axial force and mid-span deflection. The agreement is satisfactory overall, with the simulation results slightly overestimating the beam tensile axial force.
Figure 4.40: Behaviour and failure mode of Test 8
FIGURE 4.41: COMPARISON OF MODELING AND EXPERIMENTAL RESULTS FOR MID-SPAN DEFLECTION AND AXIAL FORCE IN THE BEAM RESTRAINED BY REVERSE CHANNEL JOINTS (TEST 8)

In the above 8 fire tests, the focus of each test was on structural behaviour during the heating phase, after development of some catenary action in the beam. In the next two tests, a cooling phase was included to investigate structural behaviour during both the
heating and cooling phases. Since the aim of these two tests was to assess the vulnerability of connection fracture during cooling, the two weakest connection types were investigated, being reverse channel connection with flexible endplate and fin plate connection.

4.2.2.9 Test 9: fin plate connection (including a cooling phase)

Test 9 used CHS 193.7×5mm tubes and fin plate connection. Referring to Figure 4.42, a fin plate of 8mm thickness was welded to the middle of the tubular wall with FW 8mm on both sides and bolted to the beam with two M20 Grade 8.8 bolts. There was no fire protection on the joints. During the test, the furnace fire exposure was stopped and fan-assisted cooling started just when the beam was about to change its behaviour from bending to catenary action.

Figure 4.43 shows the FE mesh and Figure 4.44 presents the input temperature-time relationships for the beam web and flanges, the steel tube, the connection zone and the concrete fill. In the numerical model, the yield strength, ultimate strength and elastic modulus of the steel members at ambient temperature were obtained from tensile coupon tests and they are given in Table 4.7. For Test 9 the average concrete cube strength was 45.3 MPa and the density was 2212 kg/m³.

Figure 4.42: Geometrical details of fin plate connection in Test 9 of Ding & Wang (2007)
Table 4.7: Mechanical property values for different steel members in Test 9

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>Fin plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>206606</td>
<td>206378</td>
<td>200001</td>
<td>190386</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>352</td>
<td>309</td>
<td>406</td>
<td>311</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>493</td>
<td>477</td>
<td>539</td>
<td>468</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>32.1</td>
<td>34.9</td>
<td>29.5</td>
<td>37.2</td>
</tr>
</tbody>
</table>

Figure 4.43: FE mesh for Test 9

(a) Before 60 min.
Figure 4.44: Time-temperature relationships used in Test 9

Figure 4.45 compares the modelling and experimental results for deformation modes of the beam and the joint. It can be seen that the observed deformations of the joint components and the beam were closely followed by the numerical model. As shown in Figure 4.45(a), there was no fracture and failure of the sub-frame. Due to expansion and bending of the steel beam, the bottom flange of the beam was bearing against the CFT columns during the heating stage, as shown in Figure 4.45(a) & (b). Apart from these, there was no noticeable deflection in the steel tubes. Due to large twist in the beam, the fin plates were also twisted to one side as shown in Figure 4.45(b).
Figure 4.45: Deformation pattern of Test 9 structure
Figure 4.46: Comparison of modeling and experimental results for mid-span deflection and axial force in the beam restrained by fin plate connections (Test 9)
Figure 4.46 compares the measured and simulated beam axial force and beam mid-span deflection as functions of the beam lower flange temperature at mid-span. The test structure experienced high compression but small lateral deflection during the thermal expansion stage. Because cooling started just after the beam reached its limiting temperature for bending, the beam also developed high tension forces during the cooling stage. The simulation results have accurately captured the entire cycle of beam behaviour.

4.2.2.10 Test 10: reverse channel connection with flexible endplate (including a cooling phase)

Test 10 used CHS 193.7×5mm tubes and reverse channel connection. Referring to Figure 4.47, the flanges of a reverse channel section 152×89 were welded to the tubular wall with FW 8mm on the outside. A flexible endplate of 8mm thickness was welded to the beam with FW 6mm on both sides and then bolted to the reverse channel with four (2 by 2) M20 Grade 8.8 bolts. There was no fire protection on the joints.

Figure 4.48 shows the FE model and Figure 4.49 presents the input temperature-time relationships for the beam web and flanges, the steel tube, the connection zone and the concrete fill. Table 4.8 lists the average yield strength, ultimate strength and elastic modulus of the steel members at ambient temperature used in the numerical model, based on steel tensile coupon tests. For Test 10 the average concrete cube strength was 46.2 MPa and the concrete density was 2305 kg/m³.
CHAPTER 4 – VALIDATION OF THE NUMERICAL SIMULATION MODEL

Figure 4.47: Geometrical details of reverse channel connection in Test 10 of Ding and Wang (2007)

Figure 4.48: FE mesh for Test 10

Table 4.8: Mechanical property values for different steel members in Test 10

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>End plate</th>
<th>Reverse channel web</th>
<th>Reverse channel flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>206606</td>
<td>206378</td>
<td>200001</td>
<td>-</td>
<td>192280</td>
<td>207811</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>352</td>
<td>309</td>
<td>406</td>
<td>-</td>
<td>397</td>
<td>351</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>493</td>
<td>477</td>
<td>539</td>
<td>-</td>
<td>562</td>
<td>561</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>32.1</td>
<td>34.9</td>
<td>29.5</td>
<td>-</td>
<td>27.1</td>
<td>32.6</td>
</tr>
</tbody>
</table>
Figure 4.50 compares the modelling and experimental results for deformation modes of the beam and the joint. It can be seen that the deformation patterns obtained by the simulation and from the test are very close. No failure was observed in both the test
specimen and numerical model. The beam was bent slightly, as shown in Figure 4.50 (a). There was no visible deformation in the steel tube and the connection region, as shown in Figure 4.50 (b). Figure 4.51 shows that both the numerical and measured beam axial force match well.

The test structure experienced large compression during the thermal expansion stage. However, unlike Test 9, because in this test cooling started just before the beam reached its limiting temperature in bending, the beam was still in substantial compression when cooling started. As a result, the residual tensile force in the beam was quite small when compared with Test 9. This is closely predicted by the FE model.

Figure 4.50: Behaviour of Test 10
Figure 4.51: Comparison of modeling and experimental results for mid-span deflection and axial force in the beam restrained by reverse channel connection (Test 10)
4.3 CONCLUSIONS

This chapter presented comparisons between the author’s numerical simulation results and Gillie’s benchmark results, and between the author’s finite element simulations and the 10 fire tests recently conducted in the University of Manchester on restrained steel subassemblies using four different types of beam to column joint. Two of the tests included both heating and cooling phases. The following conclusions may be drawn:

1. The author’s numerical simulations were able to follow Gillie’s benchmark results and reveal the various complex structural behaviour phenomena.

2. The proposed finite element models give very good agreement with the experimental results and observations, for the deformed shapes and failure modes, and for the measured beam horizontal axial force – temperature and beam vertical deflection – temperature relationships.

3. Due to the accuracy of the FE model for reproducing the experimental behaviour of steel joints to CFT columns in fire conditions, the proposed FE model can be used for parametric studies as a reliable and cost-effective tool, which may lead to improvements in joint configuration and joint performance.
CHAPTER 5 – METHODS OF IMPROVING THE SURVIVAL TEMPERATURE IN FIRE OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION

5.1. INTRODUCTION

This chapter will present the results of a numerical study, using ABAQUS, of the behaviour and methods of improving the survival temperatures in fire of steel beams to concrete filled tubular (CFT) columns using reverse channel connection. The beams are axially restrained by the connected columns and develop catenary action so their survival temperatures are primarily controlled by the joint tensile resistance and deformation capacity. Therefore, improving the beam survival temperature mainly relies on improving the joint performance. Five different joint types of reverse channel connection have been investigated: extended endplate, flush endplate, flexible endplate, hybrid flush/flexible endplate and hybrid extended/flexible endplate. The connection details investigated include reverse channel web thickness, bolt diameter and grade, using fire-resistant (FR) steel for different joint components (reverse channel, end plate and bolts) and joint temperature control. The effects of changing the applied beam and column loads are also have been considered.

5.2. SIMULATION METHODOLOGY

Figure 5.1 shows the structural arrangement to be simulated in this research. It represents a steel beam connected to two concrete filled tubular (CFT) columns. The top and bottom of the columns are rotationally unrestrained but are horizontally restrained to simulate the lateral stability system in a real structure. This structural arrangement is the same as used in the fire tests of Ding and Wang (2007). The simulation methodology was presented in Chapter 3 and validated in Chapter 4.
The following conditions apply to the simulation structure;

- Boundary conditions: half of the structure was modelled due to symmetry. The bottom of the columns was pinned in all three directions and the top of the columns was pinned in two directions but movement along the column axis was allowed; because half of the structure was modelled due to symmetry, all nodes at the beam mid-section were fixed in the axial direction, which effectively prevented rotation about the two principal axes of the beam cross-section, but allowed the beam to twist about its longitudinal axis. To represent the effect of the concrete slab, the beam was assumed to be fully restrained in the lateral direction.

- The reduction factors for strength and elastic modulus of carbon steel at elevated temperatures were the same as in EC3 (EN 1993-1-2).

- The loads were applied in two steps: (i) two point structural loads were applied to the beam at ambient temperature; (ii) while maintaining the structural loads, the structural temperatures were increased until structural failure.

- In the FE model, six different temperature curves were adopted for the different parts of the structure, as shown in Figure 5.2; a total of three temperature curves for the bottom flange, web and top flange of the beam; one temperature curve for the joint zone which included all the bolts, nuts and connection components; one temperature curve for the steel tubular column in the joint region (90 cm); one temperature curve for concrete fill in the joint region (90 cm). The temperature of the other two parts away from the joint zone was set at ambient temperature. The time-temperature curves were based on the time-temperature curves of test 4 of the Ding and Wang (2007) fire tests. In their test, the structural assembly failed at 30 min so the time-temperature curves adopted in this parametric study were extended proportionally and artificially for the bottom flange of the beam to reach 1000°C at 60 min, as shown in Figure 5.2. According to Annex D of EN 1993-1-2, the temperatures in the joint region at different heights may be assumed to be proportional to that of the bottom flange of the beam at mid span. In this research, the temperatures in the joint were assumed to be uniform, being
the average over the connection height, which gives 0.82 times the temperature at the bottom flange of the beam at mid span.

- The survival temperature of the beam was defined as the bottom flange temperature at which the beam failed to support the applied load.

![Figure 5.1: Dimensions and boundary condition of structure assembly](image-url)
CHAPTER 5 – METHODS OF IMPROVING THE SURVIVAL TEMPERATURE IN FIRE OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION

![Figure 5.2: Assumed time-temperature curves for parts of the structural arrangement](image)

5.3. PARAMETRIC STUDIES

To investigate methods of developing reverse channel connection to CFT columns to enhance catenary action in the connected beam, an extensive set of numerical parametric studies was carried out. The reference parameters were:

- Beam section size: 457x152x67UB (flange width 153.8mm, overall height 458mm, flange thickness 15mm, web thickness 9mm);

- CFT column size: Square Hollow Section 300×12.5mm;

- Beam span: 10m; the total column height: 8m (2 storeys of 4m);

- Channel section size: 230×90 (overall depth 230mm, overall width 90mm, leg thickness 14mm, web thickness 7.5mm);

- Endplate thickness: 10 mm;

- Material properties: the stress-strain constitutive relationships adopted in the FE models for the steel beams, columns and connection components were based on the steel tensile coupon tests at ambient temperature (Table 5.1) of test 4 of the
fire test results of Ding and Wang (2007). For ABAQUS simulation, the nominal engineering stress-strain model obtained from the steel tensile coupon test was converted to the true stress-strain relationship;

- Temperature profiles: see Figure 5.2, based on extending the time-temperature curves of test 4 of the Ding and Wang (2007) fire tests.

- The applied load ratio is defined as the ratio of the maximum bending moment of the simply supported beam to the plastic moment capacity of the beam at ambient temperature. In the parametric study, the base load ratio was 0.4.

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>End plate; Reverse channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>210050</td>
<td>226690</td>
<td>203210</td>
<td>210000</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>396</td>
<td>379</td>
<td>492</td>
<td>355</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>550</td>
<td>572</td>
<td>536</td>
<td>560</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>25.3</td>
<td>27.2</td>
<td>19.4</td>
<td>27</td>
</tr>
</tbody>
</table>

The numerical studies investigated the effects of the following parameters:

- Connection configuration: during catenary action, the connection is subjected to hogging bending moment and tension from the connected beam, putting the topmost connection component under the maximum tensile force. Deformation capacity requires the connection to be as flexible as possible and connection resistance requires the topmost components to be as strong as possible. Different connection configurations may achieve different combinations of strength and deformation capacity and this parametric study investigated the following five configurations: flexible endplate (Figure 5.3a), hybrid flush/flexible endplate (Figure 5.3b) with flush endplate at the top providing tensile resistance and flexible endplate at the bottom to enhance deformation capacity, hybrid extended/flexible endplate (Figure 5.3c) with the same reasoning as for a hybrid
flush/flexible endplate; flush endplate (Figure 5.3d) and extended endplate (Figure 5.3e);

- Reverse channel web thickness: in the fire tests by Ding and Wang (2007), test 8 using a flush endplate reverse channel connection failed by fracture of the front face of the reverse channel. Due to shear fracture of the reverse channel, the failure mode was brittle and therefore, the catenary action phase was very short. This parametric study investigated the effect of increasing the thickness of the reverse channel web to prevent its fracture and achieve a prolonged phase of catenary action;

- Bolt grade & bolt diameter: in reverse channel connections the bolts must be strong enough to prevent failure of the connection due to bolt fracture under combined bending and tension. In this parametric study two bolt grades 8.8 and 10.9 were used to investigate the effect of bolt strength. Three bolt diameters M20, M24 and M27 were also examined. Table 5.2 shows the ambient temperature properties used for bolt grades 8.8 and 10.9;

Table 5.2: Mechanical property values for bolts at ambient temperature

<table>
<thead>
<tr>
<th>Component</th>
<th>Grade 8.8</th>
<th>Grade 10.9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>210000</td>
<td>210000</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>640</td>
<td>900</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>800</td>
<td>1000</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>16</td>
<td>16</td>
</tr>
</tbody>
</table>
CHAPTER 5 – METHODS OF IMPROVING THE SURVIVAL TEMPERATURE IN FIRE OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION

(a) Flexible endplate

(b) Flush endplate
CHAPTER 5 – METHODS OF IMPROVING THE SURVIVAL TEMPERATURE IN FIRE OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION

Figure 5.3: Basic geometrical details of 5 configurations of reverse channel connection
- Joint fire protection regime: since joint components are the key in controlling structural failure during the beam catenary action phase, this study investigated methods of changing the relative strength and ductility of the joint components relative to the beam by varying the joint fire protection (temperature) regime.

- Fire resistant (FR) steel: another method of potentially increasing joint performance is to use fire resistant steel which could achieve better retention in strength and better ductility at elevated temperatures compared to normal carbon steel. In this parametric study, the reduction factors for FR steel at elevated temperatures were according to those suggested by Kelly and Sha (1999) (as shown in Figure 5.4). However, the elevated temperature properties of FR bolts were from Sakumoto et al (1993). Table 5.3 compares the FR bolt properties with those of bolts made of conventional carbon steel.

- Deformation capacity of steel: the mechanical properties of steel that are currently in use (CEN 2005b) were based on experimental results of many years ago (e.g. Kirby (1995)). At the time, since the emphasis of the tests was on conventional bending/axial resistance, there was no thorough investigation of the deformation capacity (ultimate strain) of the steel. In order to investigate the feasibility of developing robust steel structures in fire, this research will assess the effects of adopting different ultimate strain values for steel, ranging from the stress-strain relationships of steel in EN 1993-1-2 (shown in Figure 5.5) to artificial stress-strain curves of steel with 30% additional strain as shown by the dotted lines in Figure 5.5.
### Table 5.3: Strength reduction factors and elongation % for FR steel bolts grade 10.9 from Sakumoto et al (1993)

<table>
<thead>
<tr>
<th>Temperature °C</th>
<th>Carbon steel bolts</th>
<th>FR bolts</th>
<th>FR bolts</th>
<th>FR bolts</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Tensile strength</td>
<td>Young’s modulus</td>
<td>Elongation %</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>14</td>
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<tr>
<td>300</td>
<td>0.903</td>
<td>0.959</td>
<td>0.898</td>
<td>16</td>
</tr>
<tr>
<td>400</td>
<td>0.775</td>
<td>0.874</td>
<td>0.896</td>
<td>13</td>
</tr>
<tr>
<td>500</td>
<td>0.550</td>
<td>0.747</td>
<td>0.790</td>
<td>13</td>
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<tr>
<td>550</td>
<td>0.385</td>
<td>0.624</td>
<td>0.716</td>
<td>17</td>
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<tr>
<td>600</td>
<td>0.220</td>
<td>0.430</td>
<td>0.608</td>
<td>23</td>
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<td>650</td>
<td>0.165</td>
<td>0.273</td>
<td>0.444</td>
<td>34</td>
</tr>
<tr>
<td>700</td>
<td>0.100</td>
<td>0.166</td>
<td>0.333</td>
<td>57</td>
</tr>
<tr>
<td>800</td>
<td>0.067</td>
<td>0.074</td>
<td>0.234</td>
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</tr>
<tr>
<td>900</td>
<td>0.033</td>
<td>0.033</td>
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<tr>
<td>1000</td>
<td>0</td>
<td>0</td>
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<td>-</td>
</tr>
</tbody>
</table>

**Figure 5.4: Strength reduction factors for the FR steels and the carbon steel from Kelly and Sha (1999)**
CHAPTER 5 – METHODS OF IMPROVING THE SURVIVAL TEMPERATURE IN FIRE OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION

Figure 5.5: Stress-strain relationships of steel at elevated temperatures: EN 1993-1-2 values and artificial values of a “ductile” steel

5.4. PARAMETRIC STUDY RESULTS

Tables from 5.4 to 5.8 list all the simulations carried out in the parametric study. In this section, the simulation results will be presented by considering the effects of the six parameters identified in the previous section: (1) connection configuration; (2) reverse channel connection web thickness; (3) bolt grade and bolt diameter; (4) joint fire protection scheme (temperature); (5) FR steel for connection components, and (6) ductility (ultimate strain) of steel. The tables also compare the beam’s limiting temperature, the mode of failure and the survival temperatures of all the simulations, the survival temperature being defined as the maximum steel beam lower flange temperature above which the applied load on the structure could not be sustained.
## Table 5.4: Parametric study results for extended endplate connection

<table>
<thead>
<tr>
<th>Simulation ID</th>
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<th>3</th>
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<th>5</th>
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<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
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<tbody>
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<td>10</td>
<td>10</td>
<td>12</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Revere channel web thickness (mm)</td>
<td>7.5</td>
<td>9</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>13</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>Bolt grade</td>
<td>8.8</td>
<td>8.8</td>
<td>8.8</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
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<td>M20</td>
<td>M20</td>
</tr>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>CFP</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Use of Fire Resistant Steel</td>
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<td>-</td>
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<tr>
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<td>716</td>
<td>718</td>
<td>727</td>
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<td>825</td>
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<tr>
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<td>547</td>
<td>563</td>
<td>609</td>
<td>645</td>
<td>600</td>
<td>677</td>
<td>773</td>
<td>-</td>
</tr>
<tr>
<td>Limiting temperature (°C)</td>
<td>710</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Failure mode</td>
<td>channel web</td>
<td>channel web</td>
<td>bolts</td>
<td>bolts</td>
<td>bolts</td>
<td>bolts</td>
<td>beam</td>
<td>channel web &amp; endplate</td>
<td>bolts</td>
<td>-</td>
</tr>
</tbody>
</table>

BFRS= bolts using fire resistant steel  
CFRS= connection components using fire resistant steel  
CFP= connection fire protected (upper limits on joint component temperatures)
### Table 5.5: Parametric study results for extended/flexible endplate connection

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<td>10</td>
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<tr>
<td>Revere channel web thickness (mm)</td>
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<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>13</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>Bolt grade</td>
<td>8.8</td>
<td>8.8</td>
<td>8.8</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
</tr>
<tr>
<td>Bolt diameter (mm)</td>
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<td>M20</td>
<td>M20</td>
<td>M20</td>
<td>M24</td>
<td>M27</td>
<td>M27</td>
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<td>M20</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>CFP</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Use of Fire Resistant Steel</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>BFRS</td>
<td>BFRS</td>
<td>CFRS</td>
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<td>-</td>
<td>-</td>
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<td>-</td>
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<td>668</td>
<td>600</td>
<td>-</td>
<td>753</td>
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<td>-</td>
<td>bolts</td>
<td>bolts</td>
<td>bolts</td>
<td>beam</td>
<td>-</td>
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Table 5.6: Parametric study results for flexible endplate connection

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<tbody>
<tr>
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<td>10</td>
<td>12</td>
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</tr>
<tr>
<td>Revere channel web thickness (mm)</td>
<td>7.5</td>
<td>9</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>13</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>Bolt grade</td>
<td>8.8</td>
<td>8.8</td>
<td>8.8</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
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<tr>
<td>Bolt diameter (mm)</td>
<td>M20</td>
<td>M20</td>
<td>M20</td>
<td>M20</td>
<td>M24</td>
<td>M27</td>
<td>M27</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>CFP</td>
<td>-</td>
<td>-</td>
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</tr>
<tr>
<td>Use of Fire Resistant Steel</td>
<td>-</td>
<td>-</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>BFRS</td>
<td>BFRS</td>
<td>CFRS</td>
<td></td>
</tr>
<tr>
<td>Bottom flange temp (°C)</td>
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<td>bolts</td>
<td>bolts</td>
<td>-</td>
<td>channel web &amp; endplate</td>
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<td>beam web</td>
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### Table 5.7: Parametric study results for flush endplate connection

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<td>10</td>
<td>10</td>
<td>12</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Revere channel web thickness (mm)</td>
<td>7.5</td>
<td>9</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>11</td>
<td>13</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>Bolt grade</td>
<td>8.8</td>
<td>8.8</td>
<td>8.8</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td></td>
</tr>
<tr>
<td>Bolt diameter (mm)</td>
<td>M20</td>
<td>M20</td>
<td>M20</td>
<td>M24</td>
<td>M27</td>
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<td>BFRS</td>
<td>BFRS</td>
<td>CFRS</td>
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<td>web around bolts holes</td>
<td>web around bolts holes</td>
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<td>web around bolts holes</td>
<td>bolts</td>
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### Table 5.8: Parametric study results for flush/flexible endplate connection

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</tr>
<tr>
<td>Revere channel web thickness (mm)</td>
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<td>11</td>
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<td>11</td>
<td>11</td>
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</tr>
<tr>
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<td>8.8</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
<td>10.9</td>
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<tr>
<td>Bolt diameter (mm)</td>
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<td>M20</td>
<td>M20</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>CFP</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Use of Fire Resistant Steel</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>BFRS</td>
<td>BFRS</td>
<td>CFRS</td>
<td></td>
</tr>
<tr>
<td>Bottom flange temp (°C)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>721</td>
<td>-</td>
<td>750</td>
<td>-</td>
<td>-</td>
<td>753</td>
<td>753</td>
</tr>
<tr>
<td>Connection temp (°C)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>554</td>
<td>-</td>
<td>603</td>
<td>-</td>
<td>-</td>
<td>607</td>
<td>706</td>
</tr>
<tr>
<td>Limiting temperature (°C)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>705</td>
</tr>
<tr>
<td>Failure mode</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>web around bolts holes</td>
<td>-</td>
<td>web around bolts holes</td>
<td>-</td>
<td>-</td>
<td>web around bolts holes</td>
<td>Bolts &amp; web around bolts</td>
</tr>
</tbody>
</table>
5.4.1. Effect of joint configuration

Figure 5.6 compares the effects of using the following different joint configurations: flexible endplate, extended endplate, hybrid extended/flexible endplate, flush endplate and hybrid flush/flexible endplate. All the other parameters are according to simulation 6 in Tables 5.4-8. Owing to a slight increase in axial stiffness of the extended or flush endplate connection compared to the other joint types, the beam’s compression force is slightly higher during the restrained thermal expansion stage. Also the extended or flush endplate connections have higher rotational stiffness than the other connections, so the beam’s axial buckling capacity is higher resulting in a higher maximum compression force. The temperature at which the beam’s axial force changes from compression to tension is the beam’s limiting temperature, defined by the bending resistance of the beam and the connections. Figure 5.6(a) indicates that the beams using flush, extended endplate connections and hybrid connection types reached similar, but higher limiting temperatures than the flexible endplate connection, which is expected.

The main interest of this research is the beam’s survival temperature under catenary action above the beam limiting temperature defined on the basis of the beam’s bending moment resistance. Figure 5.6(b) shows that once the beam is in the catenary action phase, the rate of the beam’s vertical deflection slows down. The beam’s survival temperature depends on the strength and ductility of the joint. Without any other modification to the joint details, the beams survival temperatures are higher than the beam’s limiting temperatures by about 40°C. This level of increase in the beam’s survival temperature is low. Nevertheless, Figure 5.6 (a) shows that using the extended or hybrid extended/flexible endplate connection resulted in better performance than using other joint types. In contrast, using the flush or hybrid flush/flexible endplate connection gave little increase in the beam’s survival temperature over the beam’s limiting temperature.
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Figure 5.6: Effects of joint type on beam behaviour, all joint components using Carbon Steel
Figure 5.7 shows the deformation patterns and failure modes of the different joints. It can be clearly seen that, the common failure mode of extended, extended/flexible and flexible endplate connections was fracture of the bolts by compound tension and bending moment, as shown in Figures 5.7 (a), (b) and (c), however, in the case of flush or flush/flexible endplate connection, the fracture of the reverse channel web around the bolt holes compound with fracture of the reverse channel web corners near the flanges, was the common mode of failure, as shown in Figures 5.7 (d) and (e).
5.4.2. Effect of reverse channel web thickness (Simulations 1, 2 and 3)

Having shown that the structural assembly may fail due to the reverse channel, this section investigates the effect of strengthening the reverse channel by increasing the reverse channel thickness. Figure 5.8 compares the simulated failure modes for three reverse channel web thicknesses: 7.5 mm, 9 mm and 11 mm, in all cases using the extended endplate connection. Increasing the reverse channel web thickness changed the connection failure mode from fracture of the reverse channel web around the bolt holes (7.5 mm) to the junction between the web and the flanges (9 mm) to the bolts (11 mm).
5.4.3. Effect of bolts grade and bolt diameter

By increasing the reverse channel web thickness, the connection failure mode moved to the bolts. One method of improving the bolt performance is to increase the bolt diameter or the bolt strength or both. In Simulation 3, the bolts were Grade 8.8 and the beam failure temperature was at 718°C. As the simulation results in Table 5.4 for Simulations 4-6 show, increasing the bolt strength grade and bolt diameter gave some increase in the beam’s survival temperature. However, this strategy is unlikely to be effective, because even when changing bolt size/grade from M20 Grade 8.8 (Simulation
3) to M27 Grade 10.9 (Simulation 6), the increase in the beam’s survival temperature was only 72°C.

When using the other joint types, changing the bolt size/grade resulted in similar changes in the beam’s survival temperature. However, when using the hybrid extended/flexible endplate connection, the beam’s survival temperatures were higher than using extended endplate connection, owing to the increased the connection deformation capacity in the compression zone.

**5.4.4. Effect of FR bolts**

In this set of simulations, bolt fracture was the governing failure mode. Therefore, if the bolt performance was improved, it would be possible to further increase the beam’s survival temperature. One method of improving bolt performance is to use Fire Resistant (FR) steel bolts. Figure 5.9 indicates that changing the bolts with FR steel bolts prevented the pull out of bolts and enabled the beam to develop a more prolonged phase of catenary action and was able to increase the beam failure temperature by 98°C from 727°C (Simulation 4) to 825°C (Simulation 8) for extended endplate connection compared to Carbon steel bolts. The failure in this case moved from the bolts to reverse channel web and the endplate, as shown in Figure 5.9 (a). In Simulation 9, the reverse channel web and the end plate thickness were increased to 13 mm and 12 mm respectively. These improvements prevented the reverse channel web and the endplate failure. This change allowed the beam to survive very high temperature at 942°C and 920°C for extended and extended/flexible endplate connections respectively. Figure 5.9 (b) shows that the connection failed by fracture of the bolts but at very high temperature. It is important to mention that, the failure temperature in the FE model was defined as the bottom flange temperature at which the beam failed to support the applied load due to fracture of some of the connection components under combined bending and tension where excessive plastic strain occurred. The ultimate strains at elevated temperatures were taken according to Table 5.3.

The results of simulation 9 of all connection types are shown in Figure 5.10. The results in Figure 5.10 show that when using extended and extended/flexible endplate
connections, the beam’s survival temperature increased by 102°C and 152°C to 920°C and 942°C respectively compared to their Carbon Steel bolt counterparts (Figure 5.6). However, when using flush or hybrid flush/flexible endplate connection, the beam’s survival temperature did not experience any improvement. In the case of flexible endplates, using FR bolts contributed in improving beam survival temperature by only about 40°C from 756°C to 795°C.

This may be explained by the different failure modes of the extended (including hybrid extended/flexible) endplate and the flush (including hybrid flush/flexible) endplate connections, which are shown in Figure 5.9. It can be clearly seen that, the common failure mode of the extended endplate connections was fracture of the bolts by combined tension and bending moment, whilst in the case of flush endplate connection, the failure mode was fracture of the reverse channel web around the bolt holes (see Figure 5.7d&e). This happened as a result of the flush endplate connection’s combination of high force (due to lower lever arm) but low ductility. The inability of flush endplate connections to develop substantial catenary action due to limited ductility was clearly observed by Ya et al. (2011) and Dia et al. (2010).

Figure 5.9: Failure modes of extended endplate connections using FR bolts
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Figure 5.10: Effects of joint type on beam behaviour all joint components using FR bolts

(a) Beam axial forces – temperature relationships (simulation 9)

(b) Beam mid-span deflections – temperature relationships (Simulation 9)
5.4.5. Effect of fire protection for the connection zone

Bolt failure was clearly critical in determining the beam survival temperature. The results in section 5.4.4 suggest that using FR bolts was able to achieve great enhancement to the beam’s survival temperature, yet the results in section 5.4.3 indicate increasing the bolt size/grade achieved much lower increases in the beam’s survival temperature. This may be explained by the rate of reduction in the bolt’s tensile strengths and the bolt’s deformation capacity. From the results of simulation 6, when extended and extended/flexible endplate connections were used, connection fracture occurred at the upper bolt row where the tensile stress was the highest. The temperature in the bottom flange of the beam was about 790°C and the corresponding connection temperature (0.82 times the beam bottom flange temperature) was about 645°C. As shown in Table 5.3, at this temperature, the conventional steel bolt retains very little (16.5%) of its ambient temperature strength but the FR bolt retains a substantial amount (27.3%) of its ambient temperature strength. Merely increasing the bolt size/grade would not be able to compensate for the substantial reduction in the bolt’s mechanical properties in the case of conventional steel bolts. Because of this, controlling the bolt (connection) temperature to not exceed the temperature above which the bolt reduces its strength substantially may achieve the same results as using FR bolts.

This is demonstrated by simulation 7 in Table 5.4, which used connection fire protection (CFP) to limit the maximum temperature in the connection (including 20 cm of the beam connected to the connection) to 600°C. In the finite element model, the connection temperatures increased to 600°C and then were kept constant while the temperature of the beam was still increasing. Figure 5.11 compares the failure mode of the protected and unprotected extended end plate connections. It can be seen that using fire protection prevented fracture of any of the connection components and moved the failure zone from the connection to the beam at the boundary between the protected and unprotected beam zone. As a result, the beam survival temperature was increased by about 125°C from 818°C to 943°C when using extended/flexible endplate connection and by about 153°C from 790°C to 943°C in case of extended endplate connection. Figure 5.12 compares the beam’s axial force – temperature and vertical deflection –
temperature relationships, clearly showing the prolonged period of catenary action development in the CFP cases.

(a) Protected connection (Simulation 7)

(b) Unprotected connection (Simulation 6)

Figure 5.11: Failure modes using extended endplate connections
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Figure 5.12: Results of mid-span deflection and axial force in the beam of protected and unprotected extended & extended/flexible endplate connections
5.4.6. Effects of using FR steel for other connection components

In section 5.5.4, it was shown that by using FR bolts in conjunction with extended and hybrid extended/flexible endplate connections, the beam’s survival temperature increased drastically. However, in case of flush and hybrid flush/flexible the increase in the beam’s survival temperature was much less. This was attributed to failure occurring in the web of the reverse channel. An attempt was made to increase the beam’s survival temperature by using FR steel for all the connection components (Simulation 10 in Table 5.4). However, the results for this simulation suggest that although using FR steel for all connection components prevented failure of the web of the reverse channel around the bolts hole; it did not increase the beam’s survival temperature significantly. This was due to bolt failure (as shown in Figure 5.13) when reaching their plastic strain limits at 756°C, as shown in Figure 5.14. In contrast, due to the increased deformation capacity and lever arm when using the extended endplate, the bolt forces were much lower at the same beam lower flange temperature, allowing the beams to survive much higher temperatures. Figure 5.14&15 compare the different maximum plastic strain of the critical connection components and the bolt forces between Simulations 4 (normal steel) and 9 (FR steel) for extended endplate connections and between Simulations 4 (normal steel) and 10 (FR steel) for flush endplate connections.

![Figure 5.13: Failure modes of flush endplate connection using FR steel (Simulation 10)](image-url)
CHAPTER 5 – METHODS OF IMPROVING THE SURVIVAL TEMPERATURE IN FIRE OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION

Figure 5.14: Plastic strains in different connection components – temperature relationships

Figure 5.15: Bolt forces – temperature relationships
5.4.7. Effects of increasing the ductility of the steel beam and bolts

The mechanical properties of steel at elevated temperatures currently in use are based on experimental research of many years ago (Kirby (1995)) when there was no interest in robustness of steel framed structures in fire. Therefore, there was little accurate information on the deformation capacity of steel at elevated temperatures. In the simulations using ordinary bolts, the maximum strain was 16% at ambient and elevated temperatures. However, for FR bolts, the variable strain limits at different temperatures in Table 5.3 were used. If ordinary bolts were to have the same deformation capacity as in Table 5.3 for FR bolts, it is expected that the beam survival temperature would be increased. This is shown in Figure 5.16(a), which compares the beam axial force – temperature relationships using extended endplate connection with different ordinary bolt deformation capacities, either constant at 16% (simulation 6) or variable (simulation 6A) as given in Table 5.3. Figure 5.16(b) further shows the increase in the beam’s survival temperature in two steps. The first step compares the effect of using FR bolt strength retention factors (Table 5.3) but with a constant strain limit of 16% (simulation 9) with using normal bolt strength reduction factor and constant strain limit of 16% (simulation 4). The second step compares the enhancement effect of using FR bolt retention factors with variable strain limits (Table 5.3), shown as simulation 9A. These comparisons clearly demonstrate the most important features of enhancing robustness of steel framed structures in fire: improving bolt ductility and strength (Simulation 6A or 9A).

In fact, by preventing bolt failure (e.g. limiting connection temperature), failure of the structure moved to the beam due to limited ductility of the steel. Currently, EN 1993-1-2 limits the steel strain at 20%. Figure 5.17 compares the simulation results for mid-span deflections and axial force in the beam if using extended endplate connections with different steel strain limits: 20% as recommended in EN 1993-1-2, 27% as obtained by Ding and Wang (2007) and an artificial level of 50%. The increases in the beam’s survival temperature were quite impressive. For example, if the steel strain limit was 50%, the steel beam survival temperature was as high as 990°C. Nevertheless, it must be said that a strain level of 50% may not be achievable for carbon steel.
CHAPTER 5 – METHODS OF IMPROVING THE SURVIVAL TEMPERATURE IN FIRE OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION

(a) Based on simulation 6, using conventional bolt strength retention factors

(b) Based on simulation 9

Figure 5.16: Effects of bolt strength retention factor and strain limit on beam axial force – temperature behaviour
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Figure 5.17: Effects of steel strain limit on beam behaviour, based on Simulation 7
5.4.8. Effects of applied load ratio

From the results obtained so far, the extended and hybrid extended/flexible endplate connections allowed the beam to achieve the highest survival temperatures. The extended endplate connection was selected to investigate the effects of applying different amounts of load on the beam. Two load levels were investigated: 0.4 and 0.7, being defined as the ratio of the applied load in fire to the beam’s ambient temperature load carrying capacity in bending with simply supported boundary conditions.

Figure 5.18 compares the simulation results. At the lower load ratio of 0.4, the beam’s limiting temperature was higher as expected. However, what is more remarkable is that at the lower load ratio, the beam was able to develop prolonged catenary action and achieve very high survival temperatures, whereas at the load ratio of 0.7, the increase in the beam’s survival temperature from the beam’s limiting temperature was quite modest. This is because at the high load ratio, the beam’s catenary force and deflections were very high. The high catenary force and deformation induced failure in all the connection components: the reverse channel web, the endplate and the bolts, as shown in Figure 5.19.

Because all connection components experienced failure when the applied load ratio was 0.7, using FR bolt (BFR) alone was not sufficient to increase the beam survival temperature greatly. More substantial increase in the beam survival temperature may be obtained by using FR steel for the connection components (CFR), as shown in Figure 5.20 where the structural failure moved from the connection to the beam. Figure 5.21 shows that in this case the beam’s survival temperature was 853°C, compared to 754°C if only FR bolts were used.
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Figure 5.18: Effects of beam load ratio on beam behaviour
Figure 5.19: Deformation pattern and failure mode of extended endplate connection with applied load ratio in the beam = 0.7, based on simulation 9

Figure 5.20: Deformation pattern and failure mode of extended endplate connection with applied beam load ratio=0.7, based on simulation 9, using FR steel for connection components
5.4.9. Effect of CFT unloaded column temperatures

The assumption of column temperature distribution was based on the authors’ modelling of the tests of Ding and Wang (2007). In realistic structures, the columns would be heated throughout the height. However, since the focus of this parametric study was on the beam and the joints with the columns having only minor influences, the assumed column temperature distribution was considered acceptable. To confirm this, Figure 5.22 compares the beam axial force and mid-span deflection–temperature curves for both column temperature distributions for simulation 9 using extended endplate which had the most prolonged stage of catenary action. The effects of using the two different temperature distributions were minimal.
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Figure 5.22: Effects of column temperature on beam behaviour, based on Simulation 9

(a) Beam axial forces – temperature relationships

(b) Beam mid-span deflections – temperature relationships
5.5. IMPLICATION ON LOADED COLUMN BEHAVIOUR

In the above simulations, all the columns were unloaded and sufficiently strong to resist the axial loads from the beam. However in reality the column will be loaded. Should catenary action be used in structural robustness design under fire, it is important that the catenary force in the beam does not cause early failure of the columns. A number of additional simulations were carried out to investigate the effects of changing the column loading and temperature distributions. The additional column temperature distributions were either heating the entire lower columns or heating all the columns (to simulate fire spread). Two additional column loading conditions were investigated: under pure axial load to give an axial load ratio of 0.5 (applied axial load = 2500 kN), under combined axial load (applied axial load=2500 kN) and maximum beam catenary force (=134.4 kN) to give a combined load ratio of about 0.5. Table 5.9 lists the different column loads, dimensions and basis of calculating the column load ratio and Figure 5.23 compares the results for these different cases, Case 1 being the base case (simulation 9 in Table 5.4). In all cases, there was no column failure due to compression load in the beam during the beam expansion stage. However, the behaviour of the beam was completely different when comparing unloaded columns with loaded columns. With unloaded columns (Case 1), the beam’s catenary action development was substantial and the beam’s survival temperature was very high. However, with load in the column but uncontrolled column temperature increase (Cases 2-5), it was not possible for the beam to develop much catenary action and the beam’s survival temperature was no more than 110°C above the beam’s limiting temperature, even when the beam’s catenary action effect was taken into consideration in calculating the column load ratio (Cases 4,5). If the beam’s catenary action force was not included in designing the column (Cases 2, 3), the increase in the beam’s survival temperature above the beam’s limiting temperature is very low (30°C). This was because the columns were not able to survive temperatures above their own limiting temperatures due to a lack of alternative load carrying mechanism to the columns. For comparison, the limiting temperatures of the two columns in Table 5.9 (Cases 2 and 4) were 560°C and 556°C respectively, similar to the column temperatures at beam failure (523°C and 572°C respectively). Column failure can be clearly seen in Figure 5.23(c) by the accelerating horizontal deformation of the beams.
If the column temperatures were controlled to be below their own limiting temperatures regardless of the increase in beam temperature, then the beam would be able to develop prolonged catenary action. This is shown in Figure 5.23 by Cases 6&7, for which the column temperature was capped at 500°C.

Table 5.9: Column loads, dimensions and temperature regimes

<table>
<thead>
<tr>
<th>Case</th>
<th>Steel tube size</th>
<th>Column load ratio</th>
<th>Na (kN)</th>
<th>Np (kN)</th>
<th>Ma (kN.m)</th>
<th>Mp (kN.m)</th>
<th>Beam catenary action force included in calculation of column load ratio?</th>
<th>Column temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SHS 300*12.5</td>
<td>Unloaded</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>90 cm connected to beam heated</td>
</tr>
<tr>
<td>2</td>
<td>SHS 300*12.5</td>
<td>Na/Np=0.5</td>
<td>2500</td>
<td>5000</td>
<td></td>
<td></td>
<td>no</td>
<td>lower column heated</td>
</tr>
<tr>
<td>3</td>
<td>SHS 300*12.5</td>
<td>Na/Np=0.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>all column heated</td>
</tr>
<tr>
<td>4</td>
<td>SHS 300*35</td>
<td>Na/Np+Ma/Mp=0.5</td>
<td>2500</td>
<td>10500</td>
<td>268.8</td>
<td>900</td>
<td>yes</td>
<td>lower column heated</td>
</tr>
<tr>
<td>5</td>
<td>SHS 300*35</td>
<td>Na/Np+Ma/Mp=0.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>all column heated</td>
</tr>
<tr>
<td>6</td>
<td>SHS 300*35</td>
<td>Na/Np+Ma/Mp=0.5</td>
<td>2500</td>
<td></td>
<td>268.8</td>
<td>900</td>
<td></td>
<td>lower column heated up to 500°C</td>
</tr>
<tr>
<td>7</td>
<td>SHS 300*35</td>
<td>Na/Np+Ma/Mp=0.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>all column heated up to 500°C</td>
</tr>
</tbody>
</table>

Na = applied axial load in column  
Ma = bending moment in the column caused by catenary action force in the beam  
Np = column axial compression resistance at ambient temperature  
Mp = column bending moment resistance at ambient temperature
CHAPTER 5 – METHODS OF IMPROVING THE SURVIVAL TEMPERATURE IN FIRE OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION

(a) Beam axial forces – temperature relationships

(b) Beam mid-span deflections – temperature relationships
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![Beam bottom flange temp. (°C)](chart.png)

(c) Beam horizontal displacement – temperature relationships

Figure 5.23: Effects of columns on beam behaviour using extended endplate connection

5.6. CONCLUSIONS

This chapter has presented the results of a numerical study using ABAQUS, to improve understanding on how different design parameters may be used to enhance the survival temperatures of steel beams connected to concrete filled tubular (CFT) columns using reverse channel connections. The design parameters investigated include the connection details (connection types, endplate/reverse channel thicknesses, bolt size/grade), connection materials (ordinary bolt and carbon steel, FR bolt, FR connection components), connection temperature regime and the beam’s steel strain limit. The investigations were carried out for beams and columns with different levels of load. The following main conclusions may be drawn.

1. Among the five connection types investigated, using the extended or hybrid extended/flexible endplate connections gave the best fire resistant performance for the beam. Using a flush endplate (or hybrid flush/flexible endplate) connection was not effective.

2. Failure in the reverse channel and the endplate can be delayed by increasing
their thickness.

3. On the other hand, using bigger or higher grade bolts would not be an effective solution. To prolong the beam’s catenary action development when the failure mode is in the bolts, using Fire Resistant (FR) bolts can be an effective solution. The main benefit comes from the FR bolt’s enhanced strength and strain limits at high temperatures.

4. Limiting the connection region temperature to be below 600°C can also have the desired effect of giving the beam very high survival temperatures.

5. Ductile materials (both the steel and bolts) are the key to achieving high beam survival temperatures. There is high uncertainty in the current steel mechanical property model which is based on experimental studies of many years ago when there was no requirement for understanding steel structural robustness in fire. Updating these mechanical property models is required.

6. The method of using catenary action to achieve high beam survival temperature is most effective when the applied load ratio in the beam is low to moderate (less than 0.5). When the applied load ratio is higher, it becomes much more difficult to devise methods to substantially increase the beam’s survival temperature above the limiting temperature. Using extended endplate connection and FR steel for all connection components offers a possible solution.

7. If using catenary action, the effect of beam axial force on the surrounding columns should be included in the column design. Very high beam’s survival temperature can be achieved in this case by limiting the column temperature below the column’s limiting temperature in combination with taking into consideration the additional bending moment in the column generated by the beam’s catenary action force.
CHAPTER 6 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION IN FIRE DURING COOLING STAGE

6.1 INTRODUCTION

During the well publicised Cardington structural fire research programme, parts of some connections suffered fracture during cooling. This led to believing that fire spread and structural collapse may occur during the cooling stage of a fire and therefore this issue should be considered in structural fire engineering design.

This and the next chapter focus on steel framed structures using concrete filled tubular (CFT) columns and the objective of these two chapters is to find means of reducing the risk of structural failure during cooling. They report the results of a study using the general finite element software ABAQUS to numerically model the behaviour of restrained structural subassemblies of steel beam to CFT columns and their joints both in fire, focusing on the cooling stage. Validation of the finite element model was presented in Chapter 4 by comparing the simulation and test results for the two fire tests investigating cooling behaviour, recently conducted at the University of Manchester on similar structures. In these two tests, the test assembly was heated to temperatures close to the limiting temperature of the steel beam and then cooled down while still maintaining the applied loads on the beam. One of the tests used reverse channel connection and the other test used fin plate connection. Remarkable differences in tensile forces in the connected beams were observed during the tests depending on the beam temperature at which cooling started. This leads to the suggestion that in order to avoid connection fracture during cooling, it may be possible to reduce the limiting
temperature of the connected beam by a small value (<50°C) from the limiting temperature calculated without considering any axial restraints in the beam.

This Chapter will focus on structural assemblies using reverse channel connection with flexible endplate and Chapter 7 will present results on fin plate connection.

The validated numerical model in chapter 4 is used to conduct extensive numerical simulations in order to investigate the behaviour of reverse channel connections between steel beams and CFT columns under cooling. The aim of this investigation is to find means of reducing the risk of joint failure during the cooling stage. Compared to flush and extended endplate connections, a flexible end plate connection is more likely to fail during cooling. Therefore, this chapter focuses on reverse channel connection using a flexible endplate. Figure 6.1 shows the structure arrangement to be simulated in this research. It represents a steel beam connected to two concrete filled tubular (CFT) columns. The top and bottom of the columns are rotationally unrestrained but are horizontally restrained to simulate the lateral stability system in a real structure. This structural arrangement is the same as used in the fire tests of Ding and Wang (2007) but the dimensions are more realistic. The beam was assumed to be fully restrained in the lateral direction to represent the effect of the concrete slab.

The results are presented in the following way: Section 6.2 presents the results of the behaviour of a basic case and Section 6.3 investigates the effects of changing different design parameters to identify feasible means of reducing occurrence of connection fracture during the cooling phase.
6.2 BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM TO CFT COLUMN USING REVERSE CHANNEL IN FIRE DURING COOLING STAGE: BASIC CASE

6.2.1 Input data for basic case

- Figure 6.2 shows details of the connection.

Other basic parameters were:

- Beam section size: 457x152x67UB (flange width 153.8mm, overall height 458mm, flange thickness 15mm, web thickness 9mm);

- Beam span: 10m;

- Total column height: 8m (2 storeys of 4m);

- Channel section size: 230×90 (overall depth 230mm, overall width 90mm, leg thickness 14mm, web thickness 7.5mm);

- Endplate thickness: 10 mm;

- Material properties: the stress-strain constitutive relationships adopted in the FE model for the steel beam, columns and connection components were based on the steel tensile coupon tests at ambient temperature (Table 6.1) of Test 4 of Ding and Wang (2007). For ABAQUS simulation, the nominal engineering stress-strain relationship obtained from the steel tensile coupon test was converted to the true stress-strain relationship;

- Figure 6.3 shows the adopted engineering strain-strain curves at different temperatures. According to EN 1993-1-2, all stress-strain curves enter the descending branches at 15% strain and completely lose stress at 20% strain;

- In the fire tests of Ding and Wang (2007), all the columns were unloaded and sufficiently strong to resist the axial loads from the beam. However, in reality the columns will be loaded. In this research, the load ratio in the columns is about 0.5, based on combined axial load and bending moment as a result of the
maximum catenary force in the connected beam. Based on this calculation, a
Square Hollow Section (SHS) 300×35 mm section was used for the columns.

- Initial applied load ratio in the beam = 0.7. Here the load ratio is defined as the
ratio of the maximum bending moment in the simply supported beam to the
plastic moment capacity of the beam at ambient temperature.

Table 6.1: Mechanical property values for different steel components at ambient
temperature

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>End plate, Reverse channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>210000</td>
<td>210000</td>
<td>203210</td>
<td>210000</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>355</td>
<td>355</td>
<td>492</td>
<td>355</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>560</td>
<td>560</td>
<td>536</td>
<td>560</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>20</td>
<td>20</td>
<td>19.4</td>
<td>20</td>
</tr>
</tbody>
</table>

Figure 6.1: Dimensions and boundary condition of structure assembly
Figure 6.2: Basic case: geometrical details of flexible end plate connection to reverse channel with flexible endplate

Figure 6.3: Mathematical model for stress-strain relationships of steel at elevated temperatures (EN 1993-1-2)
The temperature profiles (see Figure 6.4) for different parts of the structure were obtained based on the following calculation procedure:

- The ISO834 standard time-temperature curve (ISO 1980) was applied to calculate the fire temperature during the heating phase. The standard curve is given by

\[ \theta_g = 20 + 345 \log_{10} (8 t + 1) \]  

- At the maximum heating period (t_{max}), the maximum temperature in the unprotected steel beam reached its limiting temperature (585°C). The fire temperature – time relationship during the cooling phase was determined using the following equations, based on the parametric fire curve in EN 1993-1-2.

Figure 6.4 shows a set of typical time-temperature curves for different parts of the simulation structure.

\[ \theta_{g,t} = \theta_{max} - 625 (t - t_{max}) \quad \text{for } t_{max} \leq 0.5 \]  

\[ \theta_{g,t} = \theta_{max} - 250 (3 - t_{max}) (t - t_{max}) \quad \text{for } 0.5 < t_{max} < 2.0 \]  

\[ \theta_{g,t} = \theta_{max} - 250 (t - t_{max}) \quad \text{for } 2.0 \leq t_{max} \]  

Where;

t is time,

\( \theta_{g,t} \) is the gas temperature at time t;

\( \theta_{max} \) is the value of the gas temperature at the end of the heating phase;

\( t_{max} \) is the maximum heating period.

- For unprotected steel elements (beams and unprotected connections), the change in temperature \( \Delta \theta_{a,t} \) during a fire exposure time interval \( \Delta t \) should be determined from EN 1993-1-2 (2002);

\[ \Delta \theta_{a,t} = k_{sh} \frac{A_m}{c_a \rho_s} \frac{V}{h_{net,d}} \Delta t \]  

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Where;

- $k_{sh}$ is correction factor for the shadow effect (taken as 1.0 in this study);

- $A_{m}/V$ is the section factor for the steel element;

- $A_{m}$ is the surface area of the element per unit length, and $V$ is the volume of the element per unit length.

- $c_{a}$ is the specific heat of steel;

- $h_{net,d}$ is the design value of the net heat flux per unit area;

- $\Delta t$ is the time interval;

- $\rho_{a}$ is the density of steel.

- $\theta_{a,i}$ is the steel temperature at time $t$;

- $\theta_{g,i}$ is the gas temperature at time $t$;

- $\Delta \theta_{g,i}$ is the increase in gas temperature during the time interval $\Delta t$.

For protected steel elements (columns and protected connections), the change in temperature $\Delta \theta_{a,t}$ during a time interval $\Delta t$ should be determined from:

$$
\Delta \theta_{a,t} = \frac{\lambda_{a} A_{p} / V \left( \theta_{g,t} - \theta_{a,t} \right)}{d_{p} c_{p} \rho_{a} \left( 1 + \phi / 3 \right)} \Delta t - (e^{\phi/10} - 1)\Delta \theta_{g,t}
$$

Where;

$$
\phi = \frac{c_{p} \rho_{p} d_{p} A_{p} / V}{c_{p} \rho_{a} A_{p} / V}
$$

Where;

- $A_{p} /V$ is the section factor of the steel section insulated by fire protection;

In which $A_{p}$ is the appropriate area of the fire protection material per unit length of the element, and $V$ is the volume of the steel element per unit length.

- $c_{p}$ is the temperature dependent specific heat of the fire protection material;

- $d_{p}$ is the thickness of the fire protection material;
\( \Delta t \) is the time interval;
\( \lambda_p \) is the thermal conductivity of the fire protection system;
\( \rho_p \) is the density of the fire protection material.

- The section factor for the top flange was calculated as for a rectangular section exposed to fire on three sides using the following formula;

\[
A_m / V = \frac{b + 2t_f}{b \cdot t_f} \quad \text{---} 6.8
\]

Where;

- \( b \) is the flange width
- \( t_f \) is the flange thickness

- For the reverse channel web and the endplate, Ding and Wang (2009) suggested that the section factor may be calculated as for a steel plate with the combined thickness, i.e. \( 2/(t_1 + t_2) \).

Where;

- \( t_1 \) is the end plate thickness;
- \( t_2 \) is the reverse channel web thickness.

- The tubular column was assumed to be fully protected. The steel tube in a CFT column may be treated as an empty tube for the purpose of calculating the steel tube temperature (Wang and Orton (2008)). The equivalent steel tube thickness may be calculated as follows:

\[
t_{se} = t_s + t_{ce}
\]

with

\[
t_{ce} = 0.15b_i \quad \text{for } b_i \text{ (or } d_i) < 12\sqrt{T} \quad \text{---} 6.9
\]

or

\[
t_{ce} = 1.8\sqrt{T} \quad \text{for } b_i \text{ (or } d_i) \geq 12\sqrt{T} \quad \text{---} 6.10
\]
where:

- $t_s$ is the original steel tube thickness (mm)
- $t_{ce}$ is the increase in steel tube thickness for temperature calculation (mm);
- $b_i$ is the minimum dimension of the concrete core (mm)
- $T$ is the fire resistance time (min).

![Diagram of temperature and time relationships during fire exposure for a steel beam connected to a CFT column using reverse channel connection. The diagram shows temperature variations for different components such as gas (fire) temperature, beam (bottom flange & web), connection, beam (top flange), and steel tube over time. The graph is labeled (a) Before 60 min.]
Figure 6.4: Input time-temperature curves for different parts of the simulation structure

6.2.2 Simulation results

Figure 6.5(a) shows axial force developments in the beam with continuous heating only and with heating up to the beam’s limiting temperature followed by cooling. At the beginning of fire exposure, due to restrained thermal expansion, an axial compression force is present in the beam and the compression force increases with increasing temperature until reaching the maximum value (113 kN) at 437°C. Afterwards, the beam mid-span deflection starts to increase more rapidly until reaching 658 mm (more than span/20) at the maximum beam temperature of 585°C (the beam’s limiting temperature), as shown in Figure 6.5(b). For the beam with continuous heating, failure occurs at 618°C after some catenary action has developed in the beam. For the beam in cooling, the beam deflection changes within a narrow range because the beam deflection is mainly plastic. However, due to restrained cooling, the beam develops tension force at decreasing temperature. Eventually, the connection fails at near ambient temperature (22°C) when the beam tension force reaches about 168 kN, as a result of excessive plastic strains (larger than 20% strain, see Fig 6.5c) in the
connection. As shown in Figure 6.6, failure is caused by fracture of the reverse channel web around the bolt holes and fracture at the reverse channel web/flange junctions.

Figure 6.7 may be used to explain the variation of plastic strain during the cooling phase shown in Figure 6.5(c). Assume a point in the structure is at temperature T1 (585°C, cooling start temperature), and its stress-strain state is at point A on the stress-strain curve at T1. On cooling, due to the change of the beam axial force from compression to tension, the stress decreases to point B and then starts to increase elastically. During this stage, although the stress within the steel increases as a result of the increasing tensile force in the connection due to restrained thermal contraction, the total strain is lower owing to increased stiffness at lower temperatures. Therefore, for a considerable period of time, the plastic strain is unchanged. Near ambient temperature (T2), the stress-strain relationships at different temperatures are almost identical. Therefore, further increase in tensile force in the connection can only be accommodated by further strain increase shown as point C in the figure. Once the strain exceeds 15% (point D), the stress-strain curve enters the descending branch and accelerated straining is necessary to maintain structural equilibrium. Connection failure occurs at 20% strain (point E).
Figure 6.5: Beam mid-span deflection – beam temperature, beam axial force – beam temperature and maximum plastic strain– beam temperature relationships
6.3 METHODS REDUCING CONNECTION FAILURE DURING COOLING: PARAMETRIC STUDIES

The simulation results for the basic case suggest that there is a risk of connection fracture during the cooling stage. A parametric study has been conducted to investigate the effects of different design parameters and how they may be changed to prevent joint failure during the cooling stage. Table 6.2 lists all the simulations carried out in the parametric study, which covered the six parameters identified in the previous section: (1) joint fire protection scheme (temperature); (2) elongation (ultimate strain) of steel; (3) reverse channel web thickness; (4) applied load ratio; (5) difference between the beam’s limiting temperature and the temperature (before reaching its limiting temperature) at which cooling starts, and (6) the beam’s axial restraint level. Table 6.2 also indicates whether the connection has failed or not.
### Table 6.2: Summary of parametric study results of reverse channel connection

<table>
<thead>
<tr>
<th>Simulation ID</th>
<th>Fire protection scheme for connection</th>
<th>Span (m)</th>
<th>Ultimate strain (%)</th>
<th>Reverse channel web thickness (mm)</th>
<th>Load ratio</th>
<th>BLT</th>
<th>Reduction in temperature (°C)</th>
<th>The beam’s axial restraint level</th>
<th>Results</th>
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<td>1</td>
<td>-</td>
<td>10</td>
<td>20%</td>
<td>7.5</td>
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<td>7.5% F</td>
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<td>F</td>
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<td>20%</td>
<td>7.5</td>
<td>0.7</td>
<td>575</td>
<td>65</td>
<td>15% NF</td>
<td>NF</td>
</tr>
</tbody>
</table>

F=Failure - NF=No Failure - CFP= Connection Fire Protected - BLT=Beam Limiting Temperature
6.3.1 Effect of fire protection scheme for the connection zone (simulations 2-4)

In the basic case, the connection zone temperature was quite high. To examine whether it is possible to prevent connection failure during the cooling stage by reducing the connection temperature, three connection fire protection (temperature) schemes were considered: the connection zone was protected by the same fire protection thickness as the beam which reached its limiting temperature at 30, 60 and 90 min of standard fire exposure respectively (fire protection (mineral wool) thickness to beam = 3.3, 8.2 and 13.7 mm respectively). Figure 6.8 shows the time-temperature curves for the protected connection zone and the beam. In all three cases, the connection zone temperature was quite low. Figure 6.9(a) compares the beam axial force – temperature relationships and Figure 6.9(b) the reverse channel strain (the reverse channel web) – beam temperature relationships for the three fire protection schemes. All the connections failed during the cooling stage by the same mode of failure as shown in Figure 6.6 (the reverse channel web). This is expected. Since connection failure occurred during the cooling stage, the heating history in the connection has little effect.

![Figure 6.8: Calculated time-temperature curves for protected and non-protected connections](image_url)
Figure 6.9: Comparison between three different fire protection schemes for beam axial force and reverse channel web strain – beam temperature relationships
6.3.2 Effects of increasing connection deformation capacity (simulation 5)

In EN 1993-1-2, the maximum strain of steel at yield stress is 15% and the fracture strain of steel is 20%. It is possible for steel to reach higher strains. For example, Ding and Wang (2007) reported fracture strain of 27% from their steel coupon tests. Since connection fracture is due to its strain limit being exceeded, it is possible to prevent connection fracture if the connection steel has higher strain capacity. To investigate this claim, a simulation was carried out in which the steel strain limits were changed from the above 15%/20% to 22%/27% respectively. Figure 6.10 shows the alternative stress-plastic strain relationships with the higher strain limits at different temperatures.

![Stress-strain relationships of steel at elevated temperatures: EN 1993-1-2 values and Ding and Wang (2007) values of a “ductile” steel](image)

Figure 6.10: Stress-strain relationships of steel at elevated temperatures: EN 1993-1-2 values and Ding and Wang (2007) values of a “ductile” steel

Figure 6.11 compares the reverse channel maximum plastic strain developments between using 20% and 27% steel strain limits. At 15%/20% strain limits, the connection fails before complete cooling to room temperature with the strain increasing at rapid rates before fracture at around 22°C. At 22%/27% strain limits, there is no connection fracture during cooling and the structure remains intact because the maximum steel strain when cooled down to the ambient temperature is still much lower.
than 27% strain above which fracture is considered to have started. Whilst the 27% strain limit is based on only one set of mechanical test results, it clearly indicates the benefits of using ductile steel and the necessity of better quantifying the strain limits of steel.

![Graph](image)

(a) Beam axial forces – beam temperature relationships

![Graph](image)

(b) Plastic strains– beam temperature relationships

Figure 6.11: Effects of steel strain limits on connection failure
6.3.3 Effect of reverse channel web thickness (simulation 6)

Because the failure mode in the previous simulation cases was reverse channel fracture, it was expected that increasing the reverse channel thickness would prolong integrity of the structure. This is confirmed. Figure 6.12 compares the deformed shape and the equivalent Mises plastic strain for two reverse channel web thicknesses: 7.5 mm and 10 mm. The thicker reverse channel (10 mm) develops much lower plastic deformation than the thin one, as shown in Figure 6.13. Increasing the reverse channel web thickness from 7.5 mm to 10 mm prevented the fracture of the reverse channel web around the bolt holes.

![Figure 6.12: Comparison of deformed shapes of reverse channel with different web thicknesses (Simulations 1 & 6)](image)

![Figure 6.13: Effects of reverse channel web thickness on connection strain-maximum plastic strain – beam temperature relationships](image)
6.3.4 Effects of applied load ratio (simulations 7&8)

In simulations 1 to 6, the applied load ratio (0.7) was high so the maximum connection plastic strain was already quite high before cooling started. The risk of connection failure during the cooling phase was high. Connection behaviour for three load levels was compared, the load ratios being 0.4, 0.5 and 0.7. Here the load ratio is defined as the ratio of the applied load in fire to the beam’s ambient temperature plastic bending moment capacity with simply supported boundary conditions.

Figure 6.14 compares the simulation results for the maximum plastic strain in the connection. Connection failure is prolonged when the load ratio is lower and at the lower load ratio of 0.4, the connection is able to survive during the cooling stage.

![Graph showing the effects of beam load ratio on connection behaviour - maximum plastic strain vs beam temperature relationships.](image)

Figure 6.14: Effects of beam load ratio on connection behaviour – maximum plastic strain – beam temperature relationships

6.3.5 Effect of the beam maximum temperatures (simulations 9-12)

Results of the above parametric studies indicate that connections may fail during the cooling stage. While it is possible to reduce this risk by changing one or more structural parameters (for example, using lower load ratio, thicker reverse channel or more ductile
steel), it is necessary to for the designer to evaluate detailed structural behaviour, which may not be available in many cases. Therefore, there is a need to find an alternative, much simpler approach. One possibility is to start cooling at a temperature lower than the beam’s limiting temperature based on bending. In an axially restrained beam, the axial force in the beam is compression at temperature lower than the limiting temperature. If the maximum beam temperature when cooling starts is lower than the beam’s limiting temperature in bending, then the axial load in the beam is compressive when cooling starts and this compression force can be used to offset the tensile force in the connection when the beam cools.

Figure 6.15 compares the beam’s axial force – beam temperature and vertical deflection – temperature relationships between three cases: the beam’s maximum temperature is equal to the beam’s limiting temperature (case 1), the beam’s maximum temperature is 10°C less than the beam’s limiting temperature (case 2) and the beam’s maximum temperature is 25°C less than the beam’s limiting temperature (case 3). From Fig, 26(a), it can be seen that a very large tension force (183kN) was generated in the beam, causing failure of the connection before it had cooled down to room temperature, Fig. 26(c) showing the maximum connection strain exceeding the strain limit of steel. Starting cooling at 10°C below the beam’s limiting temperature prolonged the connection’s survival time during cooling but the connection still failed before cooling down to ambient temperature. In contrast, because the beam in case 3 was still experiencing high compression when cooling started, the residual tension force in the beam was reduced (to 168 kN) so that the maximum connection tension strain was lower than the strain limit of steel throughout the cooling phase. Therefore, there was no connection failure in case 3.

The reduction from the beam’s limiting temperature (BLT) to the beam’s maximum temperature before cooling starts increases as the load in the beam increases because of the existing higher connection tensile strain at a higher load ratio. For example, in the case of a high load ratio (0.8), the reduction in temperature (difference between beam limiting temperature and maximum beam temperature at which cooling starts) approaches 50°C, as shown in Figure 6.16.
CHAPTER 6 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION IN FIRE DURING COOLING STAGE

(a) Beam axial force – beam temperature relationships

(b) Beam mid-span deflection – beam temperature relationships
CHAPTER 6 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION IN FIRE DURING COOLING STAGE

Figure 6.15: Effects of the beam’s maximum temperature on connection failure, BLT=Beam Limiting Temperature

Figure 6.16: Effects of load ratio on the beam’s maximum safe temperature during cooling: maximum plastic strain–temperature relationships
6.3.6 Effect of different levels of axial restraint (simulations 13-20)

The level of axial restraint is obviously an important parameter affecting beam and connection behaviour during cooling. All other conditions being the same, the tension force in the connection and the beam increases as the axial restraint stiffness increases and therefore the risk of connection failure during cooling increases. For example, Figure 6.17(c) compares the maximum connection strain at three levels of axial restraint stiffness (15%, 25% and 50% of beam axial stiffness $K_{BA}$). The maximum beam temperature when cooling starts is the same, being 50°C lower than the beam’s limiting temperature. The results in Figure 6.17 show that the connection in all three cases failed when the beam starts to cool at the beam’s limiting temperature but failure occurred at different temperatures. When the beam starts to cool at 50°C lower than the beam’s limiting temperature in case of $K_A = 0.15 K_{BA}$ there is no failure during cooling. But connection failure occurs if the axial restraint stiffness is higher. It should be pointed out that the axial restraint stiffnesses used are high compared to that in realistic design. To enable the beam with higher axial restraint stiffnesses to survive the cooling phase without a connection failure, further reductions from the beam’s limiting temperature to the maximum temperature at which cooling starts should be considered.

For example, Figure 6.18 shows that a reduction of 75°C is necessary for the case of 25% axial restraint stiffness and 125°C for the case of 50% restraint stiffness.
(a) Beam axial force – beam temperature relationships

(b) Beam mid-span deflection – beam temperature relationships
CHAPTER 6 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION IN FIRE DURING COOLING STAGE

(e) Maximum plastic strain – beam temperature relationships

Figure 6.17: Effects of axial restraint level on beam behaviour during cooling

(a) Beam axial force – beam temperature relationships
CHAPTER 6 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION IN FIRE DURING COOLING STAGE

(b) Beam mid-span deflection – beam temperature relationships

(e) Maximum plastic strain – beam temperature relationships

Figure 6.18: Effects of cooling temperature at different levels of axial restraint
6.3.7 Effect of applied load ratio with 15% axial restraint (simulations 21-28)

In order to investigate the effects of the beam applied load ratio on the behaviour of the beam and connection during cooling, different applied load ratios were applied to the 10 m beam. The applied load ratios were 0.4, 0.5, 0.6, 0.7 and 0.8. Here the load ratio is defined as the ratio of the applied load in fire to the beam’s ambient temperature plastic bending moment capacity with simply supported boundary conditions. The axial restraint stiffness was 15% of the respective beam axial stiffness $K_{BA}$. All other conditions were kept the same. Figures 19 and 20 compare the results for the beams that start cooling at different temperatures before reaching the beam’s limiting temperature, the differences being 30°C, 50°C or 60°C. Because the load ratio is different, the beam’s limiting temperatures are also different. It is clear that the beam deflection and axial force vary with the applied load ratio, having larger deflections, lower compression forces and higher tension forces at a higher load ratio, as shown in Figures 19(a)&(b) and 20(a)&(b). When the beam starts to cool at 30°C lower than the beam’s limiting temperature, beams at all applied load ratios experience connection failure during cooling. But connection failure is prevented if the beam starts to cool at 50°C lower than the beam’s limiting temperature for applied load ratios 0.5, 0.6 and 0.7, as shown in Figure19(c). More reduction in temperature is needed for 0.8 load ratio approaches 60°C, as shown in Figure 20(c).

Figure 21 summarises the simulation results, showing the required reduction in beam temperature from limiting temperature to cooling temperature for the two different axial restraint levels of 7.5% and 15%. In the case of 7.5% restraint, no reduction is required for load ratio = 0.4 and 50°C reduction is required if the load ratio is 0.8. However, in case of the higher axial restraint level (15%), more reduction in temperature is needed. Even so, if the load ratio is realistic (not exceeding 0.7), connection failure does not happen if the cooling temperature is 50°C lower than the beam’s limiting temperature.
CHAPTER 6 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION IN FIRE DURING COOLING STAGE

(a) Beam axial force – beam temperature relationships

(b) Beam mid-span deflection – beam temperature relationships
CHAPTER 6 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION IN FIRE DURING COOLING STAGE

(c) Maximum plastic strain – beam temperature relationships

Figure 6.19: Effects of cooling temperature at different beam applied load ratios: applied load ratio = 0.5 & 0.6

(a) Beam axial force – beam temperature relationships
CHAPTER 6 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION IN FIRE DURING COOLING STAGE

Figure 6.20: Effects of cooling temperature at different beam applied load ratios: applied load ratio = 0.4&0.8
6.3.8 Effects of beam span to depth ratio (simulations 29-33)

The beam span to depth ratio is another parameter which has noticeable effect on the beam and connection behaviour during cooling. A number of simulations were performed to investigate the effects of beam span to depth ratio. In these simulations, the applied load ratio was 0.7. The beam/spans were 7.5, 10 and 12.5 m, giving span depth ratios of about 17, 22 and 27 respectively. The axial restraint stiffness was 15% of the respective beam axial stiffness $K_{BA}$. All other conditions were kept the same. Figure 6.22 compares the results for the beams that start cooling at different temperatures before reaching the beam’s limiting temperature, the differences being 0°C, 50°C or 65°C. Because the relative stiffness of the axial restraint to that of the beam is the same in all beams, the initial developments of axial force in the beams are identical irrespective of the beam span. Because the load ratio is also the same, the beam’s limiting temperatures are also very close. Although the beam deflection increases with increasing beam span (Figure 6.22(b)), strains in the connections during heating are very similar (Figure 6.22(c)). During the cooling stage, the main difference is that the longer beams experience slightly more rapid increase in strain after the plateau stage.
This makes the connections to the longer beams slightly more prone to fracture. The results in Figure 6.22(c) show that if the beam starts cooling at 50°C before reaching the limiting temperature, the 7.5m and 10m beams do not experience connection failure during cooling. But connection failure occurs for the 12.5m beam span. To prevent connection failure during cooling for the 12.5m beam, cooling has to start slightly earlier, at 65°C before reaching the beam’s limiting temperature.

(a) Beam axial force – beam temperature relationships
(b) Beam mid-span deflection – beam temperature relationships

(c) Maximum plastic strain – beam temperature relationships

Figure 6.22: Effects of the beam span to depth ratio on the beam safe temperature during cooling
6.4 CONCLUSIONS

This chapter has reported the results of an intensive parametric study, using the validated numerical simulation model, to investigate different methods of reducing the reverse channel connection failure occurring during the cooling stage of a fire event. The following conclusions may be drawn:

1. There are high risks of failure in the reverse channel connection using flexible endplate during cooling. However, such failure may be prevented by increasing the reverse channel web thickness. Connection failure may also be reduced by using more ductile steel. While it would not be feasible to make more ductile steel just to enable the structure to pass the cooling stage of a fire attack without failure, more precise quantification of the strain limits of steel at elevated temperatures would help more accurate estimate of the connection’s ability to survive cooling.

2. Controlling (reducing) connection temperatures is not a very effective strategy in preventing connection fracture during cooling.

3. For beams with realistic levels of axial restraint stiffness (connection tensile stiffness < 15% of beam axial stiffness), a more effective and simple method is feasible. In this method, the beam is forced to cool at a temperature below its limiting temperature in bending. If the temperature at which beam cooling starts is lower than the beam’s limiting temperature by 50°C, the risk of connection fracture is drastically reduced. The practical implication is to design the beam for a limiting temperature that is 50°C lower than calculated without considering beam axial restraint.
CHAPTER 7 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING FIN PLATE CONNECTION IN FIRE DURING COOLING STAGE

7.1 INTRODUCTION

At ambient temperature, fin plate connections are designed to transfer shear, whereas during the late stage of fire exposure when the connected beam undergoes large deflections and in catenary action or during the cooling down stage, a tensile force is developed in the frame connection. This tensile force can be high enough to fracture the connection. Fracture of the endplate along the welds and elongation of holes in the beam web in fin plate connection were observed in the Cardington fire tests, caused by the horizontal tensile forces during cooling of the connected beam.

This chapter focuses on the behaviour of steel beams connected to CFT column in fire during cooling stage using fin plate connection. The objective of this chapter is to find means of reducing the risk of fin plate connection failure during cooling. This chapter followed the methodology reported in Chapter 6 for reverse channel connection with flexible endplate.
7.2 BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM TO CFT COLUMN USING FIN PLATE IN FIRE DURING COOLING STAGE: BASIC CASE

The structure arrangement to be simulated in this chapter is the same as the structure arrangement in chapter 6, see Figure 6.1.

Basic case input data:

- Figure 7.1 shows details of the connection;
- Fin plate: depth 350mm, overall width 100mm, plate thickness 8mm, 4M24 bolts Grade 8.8, based on the following calculation procedure according to EN 1993-1-8.
- Shear resistance of bolts under fire condition is given by:

\[ F_{v,Rd} = \frac{F_{v,\theta} \times k_{b,\theta}}{\gamma_{M2,fi}} \]  \hspace{1cm} (7.1)

and

\[ F_{v,Rd} = \frac{\alpha_v \times \sigma_{w} \times A_s}{\gamma_{Mb}} \]  \hspace{1cm} (7.2)

Where:

- \( k_{b,\theta} \): The strength reduction factor for bolts
- \( A \): Nominal area of bolt shank [mm²]
- \( A_s \): Stressed area of bolt shank [mm²]
- \( \gamma_{Mb} \): Partial safety factor for the bolt \( \gamma_{Mb} = 1.25 \)

For strength grades 4.6, 5.6 and 8.8 \( \alpha_v = 0.6 \)

For strength grades 4.8, 5.8, 6.8 and 10.9 \( \alpha_v = 0.5 \)

- The design bearing resistance of a bolt under fire condition is given by:

\[ F_{b,Rd} = \frac{F_{b,\theta} \times k_{b,\theta}}{\gamma_{M2,fi}} \]  \hspace{1cm} (7.3)

and
$F_{b, R_d} = \frac{2.5 \times \alpha_b \times f_u \times d \times t_p}{\gamma_{Mc}}$ - 7.4

Where;

$\alpha_b$ is the smallest of:

- $\frac{e_2}{3D_h}$;
- $\frac{P}{3d - 4}$;
- $\frac{f_{ub}}{f_u}$

$e_2$: The end distance
$P$: Spacing between bolts
$f_{ub}$: The ultimate strength of the bolts
$f_u$: The ultimate strength of the fin plate
$D_h$: Diameter of hole
$d$: Diameter of bolt
$t_p$: The smallest thickness of the fin plate and the beam web

- A clearance of 20 mm between the end of the supported beam and the supporting column is used to give the end of the supported beam freedom to rotate with ease before the bottom flange hits the supporting member;

- Material properties: the stress-strain constitutive relationships adopted in the FE model for the steel beam, columns and connection components are shown in Table 7.1;

- Initial applied load ratio in the beam = 0.7. Here the load ratio is defined as the ratio of the maximum bending moment in the simply supported beam to the plastic moment capacity of the beam at ambient temperature;

- The temperature profiles for different parts of the structure were obtained based on the calculation procedure adopted in chapter 6, see Figure 6.4:

- For the fin plate and the beam web, Ding and Wang (2009) suggested that the section factor may be calculated as for a steel plate with the combined thickness, i.e. $2/(t_p + t_w)$. 

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Where;

\( t_p \) is the fin plate thickness;

\( t_w \) is the beam web thickness.

**Table 7.1: Mechanical property values for different steel components at ambient temperature**

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam Web</th>
<th>Beam Flange</th>
<th>Column</th>
<th>Fin plate,</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (MPa)</td>
<td>210050</td>
<td>226690</td>
<td>203210</td>
<td>210000</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>396</td>
<td>379</td>
<td>492</td>
<td>355</td>
</tr>
<tr>
<td>Maximum strength (MPa)</td>
<td>550</td>
<td>572</td>
<td>536</td>
<td>560</td>
</tr>
<tr>
<td>Ultimate strain (%)</td>
<td>25.3</td>
<td>27.2</td>
<td>19.4</td>
<td>20</td>
</tr>
</tbody>
</table>

**Figure 7.1: Basic geometrical details of fin plate connection**
7.2.1 Simulation results for the basic case

Figure 7.2(a) shows axial force developments in the beam with heating up to the beam’s limiting temperature followed by cooling. At the beginning of fire exposure, due to restrained thermal expansion, an axial compression force is present in the beam and the compression force increases with increasing temperature until reaching the maximum value (128 kN) at 408°C. Afterwards, the beam mid-span deflection starts to increase more rapidly until reaching 508 mm (about span/20) at the maximum beam temperature of 565°C (the beam’s limiting temperature), as shown in Figure 7.2(b). For the beam in cooling, the beam deflection changes within a narrow range because the beam deflection is mainly plastic. However, due to restrained cooling, the beam develops tension force at decreasing temperature until reaching about 200 kN at ambient temperature (20°C). Due to the lower axial restraint (7.5%), no failure of fin plate connection occurred, as shown in Figure 7.2(c).

Figure 7.2(a) shows that when a higher level of axial restraint is used (15%) the maximum compression and tension force in the connection and the beam increased to 210 kN and 280 kN, respectively. Figure 7.2(c) compares the maximum connection strain for the two different axial restraint levels of 7.5% and 15%. The maximum beam temperature when cooling starts is the same, being the beam’s limiting temperature. The results in Figure 7.2 show that for the 15% restraint case, the connection fails at 50°C when the beam tension force reaches about 280 kN, as a result of excessive plastic strains (larger than 20% strain, see Figure 7.2(c)) in the connection. Figure 7.3, shows the deformed shape of the beam and plate at the end of the analyses. A significant deformation developed in both the plate and the beam web. The failure is caused by fracture of the plate due to upper bolt bearing.

When combined tension and shear failure in bolt bearing (beam web or single plate) is considered as a design criterion at ambient temperature, as in Eq. (7.4), uniform force distribution is assumed in all the bolts and the bearing capacity of the bolt group is calculated by multiplying the shear capacity of a single bolt by the number of bolts. This methodology is valid if the connection is not significantly deformed. In this study,
due to the higher plate distortion, the load in the uppermost bolt is higher than other three bolts. Figure 7.4 compares the internal forces in the different bolts and compares them with the bolt capacity for the two different axial restraint levels of 7.5% and 15%. In the case of 7.5% restraint, because the load carried by the upper bolt is less than the bearing capacity of the plate (see Figure 7.3a), no fracture in the plate occurred. However, at the higher axial restraint level (15%), the load carried by the upper bolt exceeds the bearing capacity of the plate causing failure of the plate, as shown in Figure 7.3b.

(a) Beam axial force – beam temperature relationships
CHAPTER 7 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING FIN PLATE CONNECTION IN FIRE DURING COOLING STAGE

(b) Beam mid-span deflection – beam temperature relationships

(c) Maximum plastic strain in fin plate – beam temperature relationships

Figure 7.2: Beam mid-span deflection – beam temperature, beam axial force – beam temperature and Maximum plastic strain – beam temperature relationships
Figure 7.3: Failure modes of connection
7.3 PARAMETRIC STUDIES

The above simulation results for the basic case suggest that there is a risk of connection fracture during the cooling stage. A parametric study has been conducted to investigate the effects of different design parameters and how they may be changed to prevent joint failure during the cooling stage. The parameters investigated include the stiffness of axial restraints, load ratio and depth-span ratio of the beam. Table 7.2 lists all the simulations carried out in the parametric study. Table 7.2 also indicates whether the connection has failed or not.
Table 7.2: Summary of parametric study results of fin plate connection

<table>
<thead>
<tr>
<th>Simulation ID</th>
<th>Span (m)</th>
<th>Load ratio</th>
<th>BLT</th>
<th>Reduction in temperature (°c)</th>
<th>The beam’s axial restraint level</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
<td>0.7</td>
<td>565</td>
<td>0</td>
<td>7.5%</td>
<td>NF</td>
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<td>2</td>
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<td>560</td>
<td>0</td>
<td>15%</td>
<td>F</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>0.7</td>
<td>560</td>
<td>25</td>
<td>15%</td>
<td>F</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>0.7</td>
<td>560</td>
<td>35</td>
<td>15%</td>
<td>F</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
<td>0.7</td>
<td>560</td>
<td>50</td>
<td>15%</td>
<td>F</td>
</tr>
<tr>
<td>6</td>
<td>10</td>
<td>0.7</td>
<td>557</td>
<td>50</td>
<td>25%</td>
<td>F</td>
</tr>
<tr>
<td>7</td>
<td>10</td>
<td>0.7</td>
<td>557</td>
<td>75</td>
<td>25%</td>
<td>F</td>
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<tr>
<td>8</td>
<td>10</td>
<td>0.7</td>
<td>557</td>
<td>100</td>
<td>25%</td>
<td>NF</td>
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<tr>
<td>9</td>
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<td>0.4</td>
<td>660</td>
<td>30</td>
<td>15%</td>
<td>F</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>0.4</td>
<td>660</td>
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<td>15%</td>
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<tr>
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<td>F</td>
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<tr>
<td>12</td>
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<td>40</td>
<td>15%</td>
<td>NF</td>
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<td>15%</td>
<td>NF</td>
</tr>
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<td>15</td>
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<td>0.8</td>
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<td>50</td>
<td>15%</td>
<td>F</td>
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<td>16</td>
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<td>85</td>
<td>15%</td>
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<td>17</td>
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<td>560</td>
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<td>15%</td>
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<td>18</td>
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<td>15%</td>
<td>NF</td>
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<td>19</td>
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<td>25</td>
<td>15%</td>
<td>F</td>
</tr>
<tr>
<td>20</td>
<td>10</td>
<td>0.7</td>
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<td>50</td>
<td>15%</td>
<td>NF</td>
</tr>
<tr>
<td>21</td>
<td>12.5</td>
<td>0.7</td>
<td>560</td>
<td>25</td>
<td>15%</td>
<td>F</td>
</tr>
<tr>
<td>22</td>
<td>12.5</td>
<td>0.7</td>
<td>560</td>
<td>50</td>
<td>15%</td>
<td>NF</td>
</tr>
</tbody>
</table>

F=Failure   -    NF=No Failure  
BLT=Beam Limiting Temperature

7.3.1 Effect of the beam maximum temperatures (simulations 3-5)

Results of the above parametric studies indicate that connections may fail during the cooling stage. Therefore, there is a need to find an approach to reduce this risk. One possibility is to start cooling at a temperature lower than the beam’s limiting temperature based on bending. In an axially restrained beam, the axial force in the beam is compression at temperature lower than the limiting temperature. If the maximum beam temperature when cooling starts is lower than the beam’s limiting temperature in
bending, then the axial load in the beam is compressive when cooling starts and this compression force can be used to offset the tensile force in the connection when the beam cools.

(a) Beam axial force – beam temperature relationships

(b) Beam mid-span deflection – beam temperature relationships
(c) Maximum plastic strain–beam temperature relationships

Figure 7.5: Effects of the beam’s maximum temperature on connection failure, 
BLT=Beam Limiting Temperature

Figure 7.5 compares the beam’s axial force – beam temperature and vertical deflection – temperature relationships between three cases: the beam’s maximum temperature is 25°C less than the beam’s limiting temperature (Simulation 3), the beam’s maximum temperature is 35°C less than the beam’s limiting temperature (Simulation 4) and the beam’s maximum temperature is 50°C less than the beam’s limiting temperature (Simulation 5). From Figure 7.5(a), it can be seen that a very large tension force (280kN) was generated in the beam, causing failure of the connection before it had cooled down to room temperature, Figure 7.5(c) showing the maximum connection strain exceeding the strain limit of steel. Starting cooling at 25 and 35°C below the beam’s limiting temperature prolonged the connection’s survival time during cooling but the connection still failed before cooling down to ambient temperature. In contrast, because the beam in Simulation 5 was still experiencing high compression when cooling started, the residual tension force in the beam was reduced (to 270 kN) so that the maximum connection tension strain was lower than the strain limit of steel throughout the cooling phase. Therefore, there was no connection failure in Simulation 5.
7.3.2 Effect of different levels of axial restraint (simulations 6-8)

It can be seen that the larger the axial restraints stiffness, the larger the change in axial force.

All other conditions being the same, the tension force in the connection and the beam increases as the axial restraint stiffness increases and therefore the risk of connection failure during cooling increases. For example, Figure 7.6(c) shows the maximum connection strain at a higher level of axial restraint stiffness (25% of beam axial stiffness $K_{BA}$). The maximum beam temperature when cooling starts is 50°C lower than the beam’s limiting temperature. The results in Figure 7.2 shows that when the beam starts to cool at 50°C lower than the beam’s limiting temperature in case of $K_A = 0.15 K_{BA}$ there is no failure during cooling. But Figure 7.6 shows that the connection failure occurs if the axial restraint stiffness is 25% of beam axial stiffness $K_{BA}$. Even the maximum beam temperature when cooling starts is being 75°C lower than the beam’s limiting temperature the connection failure occurs, as shown in Figure 7.6(c). It should be pointed out that the axial restraint stiffnesses used are high compared to that in realistic design. To enable the beam with higher axial restraint stiffnesses to survive the cooling phase without a connection failure, further reductions from the beam’s limiting temperature to the maximum temperature at which cooling starts should be considered. Figure 7.6 shows that a reduction of 100°C is necessary for the case of 25% axial restraint stiffness.

Figure 7.7 summarises the simulation results, showing the required reduction in beam temperature from limiting temperature to cooling temperature for the three different axial restraint levels of 7.5% 15% and 25%. In the case of 7.5% restraint, no reduction is required and 50°C reduction is required if the axial restraint levels of 15%. However, in case of the higher axial restraint level (25%), more reduction in temperature is needed, connection failure does not happen if the cooling temperature is 100°C lower than the beam’s limiting temperature.
CHAPTER 7 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING FIN PLATE CONNECTION IN FIRE DURING COOLING STAGE

(a) Beam axial force – beam temperature relationships

(b) Beam mid-span deflection – beam temperature relationships
CHAPTER 7 – BEHAVIOUR OF RESTRAINED STRUCTURAL SUBASSEMBLIES OF STEEL BEAM CONNECTED TO CFT COLUMN USING FIN PLATE CONNECTION IN FIRE DURING COOLING STAGE

(e) Maximum plastic strain – beam temperature relationships

Figure 7.6: Effects of cooling temperature at different levels of axial restraint

Figure 7.7: Effects of the beam axial restraint level on the required reduction from beam limiting temperature when cooling starts
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7.3.3 Effect of applied load ratio with 15% axial restraint (simulations 9-16)

In order to investigate the effects of the beam applied load ratio on the behaviour of the beam and connection during cooling, different applied load ratios were applied to the 10 m beam. The applied load ratios were 0.4, 0.5, 0.6, 0.7 and 0.8. Here the load ratio is defined as the ratio of the applied load in fire to the beam’s ambient temperature plastic bending moment capacity with simply supported boundary conditions. The axial restraint stiffness was 15% of the respective beam axial stiffness $K_{BA}$. All other conditions were kept the same. Figures 8 and 9 compare the results for the beams that start cooling at different temperatures before reaching the beam’s limiting temperature, the differences being 30°C, 40°C, 50°C or 85°C. Because the load ratio is different, the beam’s limiting temperatures are also different. It is clear that the beam deflection and axial force vary with the applied load ratio, having larger deflections, lower compression forces and higher tension forces at a higher load ratio, as shown in Figures 8(a) & (b) and 9(a) & (b). When the beam starts to cool at 30°C lower than the beam’s limiting temperature, beams at all applied load ratios experience connection failure during cooling. But connection failure is prevented if the beam starts to cool at 40°C lower than the beam’s limiting temperature for applied load ratios 0.5 and 0.6, as shown in Figure 8c.

The reduction from the beam’s limiting temperature (BLT) to the beam’s maximum temperature before cooling starts increases as the load in the beam increases because of the existing higher connection tensile strain at a higher load ratio. For example, in the case of a high load ratio (0.7), the reduction in temperature (difference between beam limiting temperature and maximum beam temperature at which cooling starts) approaches 50°C, as shown in Figure 7.5. More reduction in temperature is needed for 0.8 load ratio approaches 85°C, as shown in Figure 7.9(c).

Figure 7.10 summarises the simulation results, showing the required reduction in beam temperature from limiting temperature to cooling temperature for different applied load ratios. 40°C reduction is required for load ratio up to 0.6. However, in case of the higher applied load ratios, more reduction in temperature is needed. Even so, if the load ratio is
realistic (not exceeding 0.7), connection failure does not happen if the cooling temperature is 50°C lower than the beam’s limiting temperature.

(a) Beam axial force – beam temperature relationships

(b) Beam mid-span deflection – beam temperature relationships
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Figure 7.8: Effects of cooling temperature at different beam applied load ratios: applied load ratio = 0.5 & 0.6

(a) Beam axial force – beam temperature relationships

(c) Maximum plastic strain – beam temperature relationships
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(b) Beam mid-span deflection – beam temperature relationships

(c) Maximum plastic strain – beam temperature relationships

Figure 7.9: Effects of cooling temperature at different beam applied load ratios:

applied load ratio = 0.4 & 0.8
7.3.4 Effects of beam span to depth ratio (simulations 17-22)

The beam span to depth ratio is another parameter which has noticeable effect on the beam and connection behaviour during cooling. A number of simulations were performed to investigate the effects of beam span to depth ratio. In these simulations, the applied load ratio was 0.7. The beam spans were 7.5, 10 and 12.5 m, giving span depth ratios of about 17, 22 and 27 respectively. The axial restraint stiffness was 15% of the respective beam axial stiffness $K_{BA}$. All other conditions were kept the same. Figure 7.11 compares the results for the beams that start cooling at different temperatures before reaching the beam’s limiting temperature, the differences being 25°C or 50°C. Because the relative stiffness of the axial restraint to that of the beam is the same in all beams, the initial developments of axial force in the beams are identical irrespective of the beam span. Because the load ratio is also the same, the beam’s limiting temperatures are also very close. Although the beam deflection increases with increasing beam span (Figure 7.11(b)), strains in the connections during heating are very similar (Figure 7.11(c)). During the cooling stage, the main difference is that the longer beams experience slightly more rapid increase in strain after the plateau stage. This makes the
connections to the longer beams slightly more prone to fracture. The results in Figure 7.11(c) show that if the beam starts cooling at 25°C before reaching the limiting temperature, all the connections for all the beam spans experience connection failure during cooling. To prevent connection failure during cooling, cooling has to start slightly earlier, at 50°C before reaching the beam’s limiting temperature.

(a) Beam axial force – beam temperature relationships
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(b) Beam mid-span deflection – beam temperature relationships

(c) Maximum plastic strain – beam temperature relationships

Figure 7.11: Effects of the beam span to depth ratio on the beam safe temperature during cooling
7.4 Conclusions

This chapter has reported the results of an intensive parametric study, using the validated numerical simulation model, to investigate the dependability of cooling the beam at 50°C before reaching its limiting temperature without axial restraint to reduce the risk of fin plate connection failure occurring during the cooling stage of a fire event. The following conclusion may be drawn:

There are high risks of failure in the fin plate connection during cooling. However, such failure may be prevented by an effective and simple method, as reached in chapter 6 for reverse channel connection, valid for beams with realistic levels of axial restraint stiffness (connection tensile stiffness < 15% of beam axial stiffness) and realistic applied load ratios (not more than=0.7). In this method, the beam is forced to cool at a temperature below its limiting temperature without any axial restraint. If the temperature at which the beam cooling starts is lower than the beam’s limiting temperature by 50°C, the risk of connection fracture is drastically reduced. The practical implication of this method is to design the beam for a limiting temperature that is 50°C lower than calculated without considering beam axial restraint.
CHAPTER 8 – CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDIES

8.1 INTRODUCTION

The current research covers a wide range of parametric FE studies to investigate the behaviour and robustness of concrete filled tubular columns with realistic joints under fire conditions. This chapter presents a summary of the main conclusions of this study and recommendations for future studies.

8.2 FINITE ELEMENT MODEL VERIFICATION

Detailed three-dimensional finite element models, using the general finite element package ABAQUS/Standard, have been developed to simulate the behaviour of restrained structural subassemblies of steel beam to concrete filled tubular (CFT) columns and their joints in fire during heating and the cooling stage. The models incorporate nonlinear material properties, geometric non-linearity and contact interaction. The development of catenary action in the connected beams at very large deflections plays an important role in ensuring robustness of the steel framed structures in fire. Therefore, it is vital that the numerical simulations can accurately predict the structural behaviour at very large deflections. To enable this simulation, a pseudo damping factor has been introduced in the simulation model. It is important that this pseudo damping factor is not too high to render the simulation results inaccurate, but not too low so that its use to overcome numerical difficulty is made ineffective. To check whether a particular damping factor is appropriate, the simulated reaction forces can be compared with the applied loads.

The simulation method was used to model the 10 fire tests recently conducted at the University of Manchester. Validation of the model was based on comparison between the simulation and test results for the following:
Beam mid-span deflection;
Beam horizontal axial forces;
Deformation pattern and failure modes.

The numerical model for all tests was able to provide qualitative and quantitative agreement with the test results, for both the heating and the cooling stages. The validated numerical model was then used to conduct a few series of parametric studies to provide insight on how to improve robustness of CFT column assemblies under heating and cooling.

**8.3 METHODS OF IMPROVING THE SURVIVAL TEMPERATURE IN FIRE OF STEEL BEAM CONNECTED TO CFT COLUMN USING REVERSE CHANNEL CONNECTION**

Connections (joints) play the most important role to ensure integrity of the structure assembly. In the fire condition, the performance and integrity of the structural assembly is strongly influenced by strength and deformation capacity of the connections. The aim of the numerical simulations was to identify connection types and construction details that were most effective in prolonging the survival time of CFT structural assemblies. The numerical simulations focused on reverse channel connection with endplate, which has recently been identified as a suitable connection type to CFT columns.

Five different joint types of reverse channel connection were investigated:
- Extended endplate,
- Flush endplate,
- Flexible endplate,
- Hybrid flush/flexible endplate,
- Hybrid extended/flexible endplate.

The investigated connection construction details included:
- Endplate/reverse channel thicknesses, bolt size/grade,
- Connection materials (ordinary bolt and carbon steel, Fire Resistant (FR) bolt, FR connection components),
Connection temperature regime and the beam’s steel strain limit.

The following recommendations may be considered to improve the survival time of the structure by prolonging the beam’s catenary action development:

- Use extended or hybrid extended/flexible endplate connections;
- Increase the thickness of the reverse channel web;
- Using Fire Resistant (FR) bolts;
- Use fire protection for the connection zone to limit the connection region temperature below 600°C;
- Use ductile materials for the connection components (particularly bolts);
- Keep the applied load on the beam as low (load ratio < 0.5) as possible;
- Design the surrounding structure for additional loads from the connected beams;

Among the methods listed above, using Fire Resistant steel for the connection components is the most effective approach.

8.4 METHODS OF IMPROVING THE BEHAVIOUR OF STEEL BEAM CONNECTED TO CFT COLUMN IN FIRE DURING COOLING STAGE

The risk of failure in simple connections to CFT columns (reverse channel connection using flexible endplate, fin plate connection) during cooling can be substantially reduced by the following means:

- increasing the reverse channel web thickness;
- using more ductile steel for the connection components (i.e. endplate and reverse channel);

Without the need to evaluate detailed structural behaviour, it is possible to substantially reduce the risk of connection fracture during cooling by designing the steel beam for a limiting temperature that is 50°C lower than the current beam limiting temperature without considering axial restraint. This method applies to beams with realistic levels of axial restraint stiffness (connection tensile stiffness < 15% of beam axial stiffness) and load ratio (not more than 0.7).


8.5 RECOMMENDATION FOR FUTURE STUDIES

The research contained in this thesis presents detailed results of investigating means of improving structural robustness in fire through connection design. It is based on numerical investigations. Also, due to the complexity and time-consuming nature of detailed numerical simulations of structural behaviour at very large deflections, some assumptions were made and the effects of these assumptions may be further investigated in the future. A number of future research studies may be undertaken to improve understanding in this field and to encourage up-taking of the recommendations of this research.

The following future research studies may be pursued:

- The proposed FE model has ignored the welds. Further study can be performed to assess whether weld needs to be considered in the FE model.
- Experimental investigations, using the recommended methods of improving structural robustness in fire, in particular using Fire Resistant connection components, will be enormously beneficial to give confidence to these methods.
- The present study has been limited to structural assemblies (i.e. beam, columns and connection). Further detailed studies should be conducted on complete structures.
- Simplified method, incorporating realistic joint behaviour, should be developed to estimate the changes in joint forces throughout the fire exposure.
- Future investigations should be performed on more realistic arrangements using slab on top of the beam.
- More precise quantification of the strain limits of steel at elevated temperature is required.
- More robust numerical procedures should be developed to enable connection unzipping to be modelled.
- The results of FE model are based on uniform temperatures distribution along the beam length. Further study is to investigate the effects of non-uniform temperatures distribution along the beam length on the connection behaviour.
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