Behaviour of restrained structural subassemblies of steel beam to CFT column in fire during cooling stage

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A B S T R A C T

During the well publicised Cardington structural fire research programme, parts of some connections suffered fracture during cooling. This led some to believe 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 paper focuses on steel framed structures using concrete filled tubular (CFT) columns and the objective of this paper is to find means of reducing the risk of structural failure during cooling. It reports 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, emphasising on the cooling stage. Validation of the finite element model was achieved 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. The finite element models give very good agreement with the experimental results and observations. 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.

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1. Introduction

Collapse of the World Trade Center buildings [1,2] and results of the Cardington full-scale eight-storey steel framed building fire tests in the UK [3] have demonstrated that steel joints are vulnerable during both the heating and cooling phases of fire. Joint behaviour in fire is currently one of the most important topics of research on structural fire resistance; however, most of the present research studies have focused on the heating phase.

The authors [4] undertook detailed numerical simulations of the fire tests by Ding and Wang [5] to validate their numerical simulation model during the heating stage using a 3-D finite element model similar to that of Dai et al. [6]. The validated numerical model was used to conduct a comprehensive programme of parametric studies to find ways of enhancing the strength and deformation capacities of reverse channel connections to CFT columns to prolong the development of catenary action in the connected beam [7] as a means of improving structural robustness in fire.

During the cooling stage of a fire, when the beam temperature decreases, if thermal shortening of the beam is restrained, large tensile forces may be induced in the beam [8,9]. In the meantime, because of the larger thermal mass of the connection region compared to the beam, the connection temperature may still be rising so reduction in the connection strength continues. This coupling of connection strength reduction and beam tensile force increase may result in fracture of some connection components [14], leading to possible structural failure. Although this has caused concern, the authors are not aware that any systematic research has been undertaken to find ways of preventing connection failure during cooling.

This research extends the experimental study of Ding and Wang [5] and will conduct extensive numerical simulations to explore different methods of enabling reverse channel connection to survive the entire fire exposure, particularly during the cooling phase. This study has the following two specific 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 using reverse channel connections during cooling.
(2) To conduct parametric studies to find means of connection design to reduce the risk of structural failure during cooling.

This research will focus on steel structures due to their relative simplicity compared to composite steel–concrete structures. However, since the issue of joint fracture during cooling also occurred in steel–concrete composite structures, the findings of this research will be relevant to steel–concrete composite structures.

2. Validation of the numerical simulation model against the test results of Ding and Wang [5]

2.1. Description of the finite element model

Fig. 1 shows the experimental set up and details of the fire tests as provided in Ding and Wang [5]. The columns were horizontally restrained and provided axial restraints to the steel beam. As in the authors’ recent research studies [4,7], three-dimensional solid elements (C3D8) were used to model the main structural members. 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. Furthermore, to re-
Table 1
Mechanical property values for different steel members used for Test 9 of Ding and Wang [5] (fin plate connection).

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam web</th>
<th>Beam flange</th>
<th>Column</th>
<th>Fin plate</th>
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</table>

Fig. 4. Time–temperature relationships used for Test 9 of Ding and Wang [5] (fin plate connection).

Fig. 5. Deformation pattern of Test 9 of Ding and Wang [5] (fin plate connection).

Fig. 6. Comparison between modelling and experimental results for mid-span deflection and axial force in the beam with fin plate connections, Test 9 of Ding and Wang [5].
duce 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 the 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. A series of ABAQUS models were built and run to assess sensitivity of the simulation results to FE mesh. The reduction factors for strength and elastic modulus of carbon steel at elevated temperatures provided in EN 1993-1-2 [10] were used. In the test structures, many contact pairs existed in the joints. The ABAQUS contact function was used to simulate the interaction between the contact pairs. In order to reduce the computational cost, 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 been found to have little effect on the simulation results. 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 longitudinal axis was allowed. Since only half of the beam was included in the FE model due to geometrical and loading symmetry as discussed earlier, the beam mid-section was fixed in the axial direction at all nodes, which effectively prevented rotation about any axis in the beam cross-section, but allowed the beam to twist about its longitudinal axis.

![Fig. 7. Geometrical details of reverse channel connection in Test 10 of Ding and Wang [5].](image)

![Fig. 8. FE mesh for Test 10 of Ding and Wang [5] (reverse channel connection).](image)

![Fig. 9. Time–temperature relationships used for Test 10 of Ding and Wang [5] (reverse channel connection).](image)
The test beam had no lateral restraint and could move sideways. But 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. Therefore, in the numerical model the four corners of the loading plate were laterally restrained. 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 exposed to fire attack until the end of the fire test. In the FE model, six different temperature curves based on the test measurements were adopted for different parts of the structure: 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 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 concrete fill in the joint region. The temperature of the column away from the joint zone was set at ambient temperature. Since the column was not loaded, it was not necessary to refine the concrete infill model.

2.2. Finite element results

In order to evaluate reliability of the 3D finite element model, test results of two different fire tests recently conducted at the
University of Manchester [5], one using reverse channel connection and the other using fin plate connection, are used for direct comparison. The test results included deformation patterns, beam mid-span deflections and beam axial forces. As detailed in the following sections, this comparison demonstrates that the 3-D finite element model is able to successfully simulate the fire tests during both the heating and cooling stages.

2.2.1. Test 9 of Ding and Wang [5]: fin plate connection

Test 9 of Ding and Wang [5] used CHS 193.7 × 5 mm tubes and fin plate connection. Referring to Fig. 2, a fin plate of 8 mm thickness was welded to the middle of the tubular wall with 8 mm Fillet Weld (FW) 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 after the beam changed its behaviour from bending to catenary action.

Fig. 3 shows the FE mesh and Fig. 4 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 1. For the concrete in Test 9 of Ding and Wang [5], the average cube strength was 45.3 MPa and the density was 2212 kg/m³.

Fig. 5 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 Fig. 5a, 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

![Fig. 12. Dimensions and boundary condition of structure assembly.](image1)

![Fig. 13. Basic geometrical details of flexible end plate connection to reverse channel.](image2)

<table>
<thead>
<tr>
<th>Component</th>
<th>Beam web</th>
<th>Beam flange</th>
<th>Column</th>
<th>End plate, reverse channel</th>
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<td>Maximum strength (MPa)</td>
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<td>Ultimate strain (%)</td>
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shown in Fig. 5a and b. Apart from these, there was no noticeable deflection in the steel tubes. Due to large twist in the beam, the flanges were also twisted to one side as shown in Fig. 5b. Fig. 6 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.

2.2.2. Test 10 of Ding and Wang [5]: reverse channel connection

Test 10 of Ding and Wang [5] used CHS 193.7 × 5 mm tubes and reverse channel connection. Referring to Fig. 7, the flanges of a reverse channel section 152 × 89 were welded to the tubular wall with 8 mm FW on the outside. A flexible endplate of 8 mm thickness was welded to the beam with 6 mm FW 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.

Fig. 8 shows the FE model and Fig. 9 presents the input temperature–time relationships for the beam web and flanges, the steel tube, the connection zone and the concrete fill. Table 2 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 of Ding and Wang [5], the average concrete cube strength was 46.2 MPa and the concrete density was 2305 kg/m³.

Fig. 10 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 Fig. 10a. There was no visible deformation in the steel tube and the connection region, as shown in Fig. 10b. Fig. 11 shows that both the numerical and measured beam axial force match well. The numerical results for beam deflection during the heating stage are considerably higher than the experimental results. This was also the case in the previous comparison (Fig. 6). This difference in the beam mid-span deflection may be due to the difficulty in the experiment 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. To substantiate this claim, the beam’s thermal bowing deflection was calculated and the results are shown in Fig. 11b. 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 can be calculated using the following equation:

\[
\delta_b = \frac{\alpha \cdot \Delta T \cdot L^2}{h}
\]
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 (2 m) and $h$ the overall beam depth (178 mm).

The test structure experienced large compression during the thermal expansion stage. However, unlike Test 9 of Ding and Wang [5], because in this test cooling started just before the beam reached its limiting temperature in bending, the beam was still

![Fig. 16. Beam mid-span deflection–beam temperature, beam axial force–beam temperature and maximum plastic strain–beam temperature relationships.](image-url)
in substantial compression when cooling started. As a result, the residual tensile force in the beam was quite small when compared with Test 9 of Ding and Wang [5]. This is closely predicted by the FE model.

The above comparisons demonstrate that the authors' ABAQUS model is valid.

3. Behaviour of restrained structural subassemblies of steel beam to CFT column in fire during cooling stage

The validated numerical model 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. Fig. 12 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 [5] 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.

3.1. Basic case

- Compared to flush and extended endplate connections, a flexible end plate connection is more likely to fail during cooling. Therefore, this research focuses on reverse channel connection using a flexible endplate. Fig. 13 shows details of the connection.
- Other basic parameters were:
  - Beam section size: $457 \times 152 \times 67$UB (flange width 153.8 mm, overall height 458 mm, flange thickness 15 mm, web thickness 9 mm).
  - Beam span: 10 m.
  - Total column height: 8 m (2 storeys of 4 m).
  - Channel section size: $230 \times 90$ (overall depth 230 mm, overall width 90 mm, leg thickness 14 mm, web thickness 7.5 mm).
  - 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 3) of Test 4 of Ding and Wang [5]. For ABAQUS simulation, the nominal engineering stress–strain relationship obtained from the steel tensile coupon test was converted to the true stress–strain relationship.

- Fig. 14 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 [5], 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
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 4
Summary of parametric study results.

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<th>Simulation ID</th>
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<th>Span (m)</th>
<th>Ultimate strain (%)</th>
<th>Revere channel web thickness (mm)</th>
<th>Load ratio</th>
<th>BLT Reduction in temperature (°C)</th>
<th>The beam’s axial restraint level (%)</th>
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</tr>
</tbody>
</table>

F = failure, NF = no failure.
CFP = connection fire protection.
BLT = beam limiting temperature.

The temperature profiles (see Fig. 15) 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 $\theta_g = 20 + 345 \log_{10}(8t + 1)$.
At the maximum heating period \( t_{\text{max}} \), the maximum temperature in the unprotected steel beam reached its limiting temperature \( 585 \degree C \). 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 [11].

For \( t_{\text{max}} \leq 0.5 \),

\[
\theta_{\text{ct}} = \theta_{\text{max}} - 625(t - t_{\text{max}})
\]
\[ h_{g,t} = \frac{h_{max}}{C_0} \left( \frac{t}{t_{max}} \right) \text{ for } 0.5 < t_{max} < 2.0 \]

\[ h_{g,t} = \frac{h_{max}}{C_0} (t - t_{max}) \text{ for } 2.0 \leq t_{max} \]

where \( t \) is the time, \( h_{g,t} \) the gas temperature at time \( t \), \( h_{max} \) the value of the gas temperature at the end of the heating phase, and \( t_{max} \) is the maximum heating period.

- For unprotected steel elements (beams and unprotected connections), the change in temperature \( \Delta h_{t,1} \) during a fire exposure time interval \( \Delta t \) should be determined from EN 1993-1-2 [10]:

\[ \Delta h_{t,1} = k_{th} \frac{A_{ml}}{c_f \rho_a} \ln \frac{h_{net}}{d} \Delta t \]
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; in which $A_m$ 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; $q_{net,d}$ 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; $\theta_{st}$ is the steel temperature at time $t$; $\theta_{g}$ is the gas temperature at time $t$; and $\Delta \theta_{g,t}$ is the increase in gas temperature during the time interval $\Delta t$.
- For protected steel elements (columns and protected connections), the change in temperature $\Delta t_{ax}$ during a time interval $\Delta t$ should be determined from:

$$\Delta t_{ax} = \frac{\lambda_p A_p}{d_p c_p \rho_p} \frac{\theta_{ax} - \theta_{ax}}{(1 + \phi/3)} \Delta t - \left( e^{\phi/10} - 1 \right) \Delta t_{ax}$$

where

$$\phi = \frac{c_p \rho_p}{c_o \rho_o} d_p A_p / V$$

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; and $\rho_p$ 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:

$$A_m / V = \frac{b + 2t_f}{t_f}$$

where $b$ is the flange width and $t_f$ is the flange thickness.

- For the reverse channel web and the endplate, Ding and Wang [12] suggested that the section factor may be calculated as for a steel plate with the combined thickness, i.e.

$$\frac{2}{(t_1 + t_2)}$$

where $t_1$ is the end plate thickness and $t_2$ 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 [13]. The equivalent steel tube thickness may be calculated as follows:

$$t_{eq} = t_1 + t_{eq}$$

- with $t_{eq} = 0.15b_i$ for $b_i$ (or $d_i) \leq 12 \sqrt{T}$

- or $t_{eq} = 1.8 \sqrt{T}$ for $b_i$ (or $d_i) > 12 \sqrt{T}$

where $t_i$ is the original steel tube thickness (mm), $t_{eq}$ is the increase in steel tube thickness for temperature calculation (mm); $b_i$ is the minimum dimension of the concrete core (mm); and $T$ is the fire resistance time (min).

3.2. Simulation results for the basic case

Fig. 16a 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 Fig. 16b. 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 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. 16c) in the connection. As shown in Fig. 17, failure is caused by fracture of the reverse channel web around the bolt holes and fracture at the reverse channel web/flange junctions.

Fig. 18 may be used to explain the variation of plastic strain during the cooling phase shown in Fig. 16c. 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).

### 4. 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

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**Fig. 28.** Effects of axial restraint level on beam behaviour during cooling.
of different design parameters and how they may be changed to prevent joint failure during the cooling stage. Table 4 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) ductility (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 4 also indicates whether the connection has failed or not.
4.1. 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). Fig. 19 shows the time–temperature curves for the protected connection zone and the beam. In all three cases, the connection zone temperature was quite low. Fig. 20a compares the beam axial force–temperature relationships and Fig. 20b the reverse channel strain (the

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

\[ \begin{array}{c}
\text{Deflection (mm)} \\
\text{max. temp.} = \text{B.L.T-30°C-LR= 0.4} \\
\text{max. temp.} = \text{B.L.T-50°C-LR= 0.4} \\
\text{max. temp.} = \text{B.L.T-50°C-LR= 0.8} \\
\text{max. temp.} = \text{B.L.T-60°C-LR= 0.8}
\end{array} \]

\[ \begin{array}{c}
\text{Beam bottom flange temp. (°C)} \\
\text{max. temp.} = \text{B.L.T-30°C-LR= 0.4} \\
\text{max. temp.} = \text{B.L.T-50°C-LR= 0.4} \\
\text{max. temp.} = \text{B.L.T-50°C-LR= 0.8} \\
\text{max. temp.} = \text{B.L.T-60°C-LR= 0.8}
\end{array} \]

\[ \begin{array}{c}
\text{Max Strain} \\
\text{max. temp.} = \text{B.L.T-30°C-LR= 0.4} \\
\text{max. temp.} = \text{B.L.T-50°C-LR= 0.4} \\
\text{max. temp.} = \text{B.L.T-50°C-LR= 0.8} \\
\text{max. temp.} = \text{B.L.T-60°C-LR= 0.8}
\end{array} \]

\[ \begin{array}{c}
\text{Beam bottom flange temp. (°C)} \\
\text{max. temp.} = \text{B.L.T-30°C-LR= 0.4} \\
\text{max. temp.} = \text{B.L.T-50°C-LR= 0.4} \\
\text{max. temp.} = \text{B.L.T-50°C-LR= 0.8} \\
\text{max. temp.} = \text{B.L.T-60°C-LR= 0.8}
\end{array} \]

Fig. 31. Effects of cooling temperature at different beam applied load ratios: applied load ratio = 0.4 and 0.8.
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 Fig. 17 (the reverse channel web). This is expected. Since connection failure occurred during the cooling stage, the heating history in the connection has little effect.

4.2. Effects of increasing connection ductility (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 [5] 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. Fig. 21 shows the alternative stress–plastic strain relationships with the higher strain limits at different temperatures.

Fig. 22 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 cooling to room temperature with the strain increasing at rapid rates before fracture at around 36 °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.

4.3. Effects 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. Fig. 23 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 Fig. 24. 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.

4.4. Effects of applied load ratio (simulations 7 and 8)

In simulations 1–6, 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.

Fig. 25 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.

4.5. Effect of the beam maximum temperatures (simulations 9–12)

Results of the above parametric studies indicate that connection 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.

Fig. 26 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. 26a, it can be seen that a very large tension force (183 kN) was generated in the beam, causing failure of the connection before it had cooled down to room temperature. Fig. 26c 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

![Fig. 32. Effects of the beam applied load ratio on required reduction from beam limiting temperature when cooling starts.](image-url)
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

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**Fig. 33.** Effects of the beam span to depth ratio on the beam safe temperature during cooling.
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 Fig. 27.

4.6. 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, Fig. 28c compares the maximum connection strain at three levels of axial restraint stiffness (15%, 25% and 50% of beam axial stiffness Kba). The maximum beam temperature when cooling starts is the same, being 50°C lower than the beam’s limiting temperature. The results in Fig. 28 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 KA = 0.15Kba 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, Fig. 29 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.

4.7. 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 Kba. All other conditions were kept the same. Figs. 30 and 31 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 ratios are 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 Figs. 30a, b, 31a and 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 0.4, 0.5, 0.6 and 0.7, as shown in Fig. 30c. A higher reduction in temperature, about 60°C, is needed for load ratio 0.8, as shown in Fig. 31c.

Fig. 32 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.

4.8. 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 Kba. All other conditions were kept the same. Fig. 33 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 (Fig. 33b), strains in the connections during heating are very similar (Fig. 33c). During the cooling stage, the main difference is that the longer beams experience slightly more rapid strain in strain after the plateau stage. This makes the connections to the longer beams slightly more prone to fracture. The results in Fig. 33c show that if the beam starts cooling at 50°C before reaching the limiting temperature, the 7.5 m and 10 m beams do not experience connection failure during cooling. But connection failure occurs for the 12.5 m beam span. To prevent connection failure during cooling for the 12.5 m beam, cooling has to start slightly earlier, at 65°C before reaching the beam’s limiting temperature.

5. Conclusions

This paper has presented detailed simulation results of two fire tests carried out on beam to concrete filled tubular (CFT) column assemblies using reverse channel and fin plate connections for validation of the numerical simulation model during cooling stage; and reports the results of an intensive parametric study, using the validated numerical simulation model, to investigate different methods of reducing connection failure occurring during the cooling stage of a fire event. The following conclusions may be drawn:

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

2. 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’s 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.

3. For beams with realistic levels of axial restraint stiffness (connection tensile stiffness <15% of beam axial stiffness) and practical levels of loading (load ratio not exceeding 0.7), a more effective and simple method is feasible. In this method, the beam is forced to cool at a temperature below its limiting tem-
perature 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 of this conclusion is to design the beam for a limiting temperature that is 50 °C lower than calculated without considering beam axial restraint.

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