Investigation of the Post-Fire Behavior of Different End-Plated Beam–Column Connections

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Abstract: Heat affects the mechanical properties of steel and the bearing capacity of steel structures, with joints being a crucial factor in determining their behavior. Steel can regain its mechanical properties that are lost owing to heat if the temperature remains below 600 °C, allowing for the possibility of reusing steel after cooling. In such cases, it becomes essential to assess the damage caused by heat exposure to decide whether to demolish the structure or continue using it. However, continuing its usage requires anticipating the potential negative effects of heat. To achieve this, it is necessary to determine the behavior of steel joining tools experimentally or numerically after exposure to heat. This study aims to ascertain the post-fire behavior of various end-plated beam and column connections, providing a cost-effective alternative to expensive fire experiments. Three different end-plated combination models were heated to a specified temperature, and steel frames were constructed after the elements cooled. Six three-point bending tests were conducted, and the experimental data obtained were validated using finite element models. The results indicate that the temperature causes a reduction in the bearing capacity of the joint, and the length of the end plate has a significant effect on the connection behavior. The finite element model validated by experiments is expected to facilitate numerical studies with different characteristics.

Keywords: bending test; end-plated connections; finite element analysis; fire; steel

1. Introduction

The beam–column connection is the most important parameter determining the behavior of steel frames. In the design of steel structures, it is considered that these connections have certain strengths and rigidity, on the basis of which carrying capacity analysis is conducted. In recent years, the fire resistance of steel structures has been an important research topic. During fires, heat causes beam–column connections to soften, resulting in a loss of strength and rigidity, which causes a reduction in the load-carrying capacity of the related element. Many studies have been conducted to evaluate the behavior of different connection details under the effect of heat [1–8].

Many parameters affect the fire resistance of a steel column connection, with the type of connection being the most important. In general, a larger plate depth than beam depth is the most common end plate connection. A larger plate depth than beam depth increases the temperature limit by up to 45 °C. I-beams can only hold fire out for 30 min after reaching their temperature limit [9]. Beam length also affects the connection behavior in such plate connections; long beams experience high compressive forces and premature failure in the tension zones of bolts. Bolts in the tension zone fail at 500 °C for a 10 mm thick plate and at 550 °C for a 13 mm thick plate. The connection plate tends to separate more with
increasing end plate thickness [7]. Under the same end plate connection conditions, a thinner, high-strength end plate exhibits the same behavior as a thicker, soft steel plate [10,11].

Even under normal loading conditions, the resistance to fire of a durable structure is less than 30 min. This time is insufficient for a steel structure; therefore, steel structures must be protected from fire [12]. Post-fire performance evaluation is very important for steel structures because their forces and deformation are redefined after a fire [10,11].

To evaluate the behavior of end plate bolted connections in a fire, beam–column connections/frames need to be exposed to high temperatures in a furnace. However, such experiments are very expensive. Steel structures reach deflection limits between approximately 510 °C and 650 °C under normal design loads. They regain their mechanical properties when cooled below 600 °C [8,13]. After cooling, steel elements can be reused. To reuse steel elements, it is necessary to predict damage and the loss of load-bearing capacity, and the residual mechanical properties of steel elements should be investigated after a fire [14]. There are many factors that influence the behavior of steel connections, such as loading type, heating, cooling speeds, etc.

Researchers have studied the post-fire behavior of different types of steel at different temperatures [15,16]. In addition, other researchers have studied different temperature levels and steel types to investigate the reuse of steel after fires. Qiang et al. found that S460 and S690 steels were unaffected by temperatures below 600 °C, suggesting that steel could be reused after such fires. They explained that high-strength steel achieves load-bearing capacity below 550 °C, so high-strength steel could be reused after cooling below 550 °C [11].

According to Eurocode-3, after a 600 °C fire, significant decreases in mechanical properties diminish substantially. Studies indicate that steel connections may be exposed to greater deformation and crushed, even under smaller loads. [8]. Therefore, understanding the post-fire behavior of a steel structure is as important as studying its behavior during a fire to determine the availability of steel elements. Demolishing a steel structure that has been exposed to heat and rebuilding is very costly. If there is no significant reduction in the load-bearing capacity, demolishing the structure would be a waste of time and money. Therefore, the post-fire behavior of connections must be analyzed appropriately. Additionally, there is insufficient information regarding the effect of moment–rotation on end plate bolted beam–column connections without damage in post-fire conditions.

Sagiroglu heated T-beam–column steel connections to 600 °C for 5 h before cooling. After the connections reached room temperature, the column–beam connections were re-constructed and loading was continued until failure occurred. Sagiroglu emphasized that the study could be used as a basis for further research on the post-fire behavior of steel connections [8]. Cirpici et al. followed the same method [17]. Cirpici et al., 2021, investigated the experimental behavior of cold-formed stainless steel screwed beam–column connections in post-fire conditions. Only the structural beams were heated from room temperature to 600 °C for 5 h, which was thought to be the key temperature for mechanical property change, and then the members that were cooled to room temperature were statically loaded [17]. Although the durations of fires may be short, the test durations for fire moment and post-fire strength tests can be much longer. This is evident in the studies conducted in the literature. Sagiroglu [8], Cirpici et al. [17], Maali and Kilic [18], and Sagiroglu et al. [19] selected a temperature of 600 °C for a 5 h fire duration. In another study of fire resistance, the mezzanine floor of a metro station was exposed to flames for 90 min [19]. In another study, Krol and Wachowski (2021) heated Grade 8.8 tempered 32CrB3 steel bolts at different temperatures (400, 600, 800, and 1000 degrees Celsius) for durations of 60 min and 240 min [20]. In another study, Maali vd. 2023 investigated the post-fire behavior of screw column–beam joints using fire-protective intumescent paint on frames made of cold-formed steel structures, where the specimens were exposed to fire for two hours according to the standard fire curve [21]. As mentioned above, steel regains its mechanical properties after cooling below 600 °C. During the fire, the average ambient temperature rises to approximately 500 °C within the first 5 min, to around 600 °C after
10 min, to approximately 700 °C after 15 min, and to about 800 °C after 30 min, with the temperature increasing more slowly as time progresses. The fire can be extinguished as soon as it starts before reaching very high temperatures or alternatively, the steel can be insulated against fire. Therefore, determining whether a steel structure can be reused is significant, as demolishing and rebuilding or unnecessary strengthening are more expensive. If the post-fire behavior of connections can be predicted, damaged parts of the structure would be replaced by new ones in favor of demountable steel structures instead of demolishing the structure. To present the post-fire behavior of the connection, similarly, demountable connection elements could be heated instead of all elements, because fire experiments are too expensive. The aim of this study is to investigate the post-fire behavior of commonly used end plate bolted connections in practice through a lower-supply experimental study.

This study aims to investigate the post-fire performances of end plate bolted connections with different geometric properties. For this purpose, three different end plate connection configurations have been designed. Since fire tests are very costly, only the connection elements were heated 600 °C, which is indicated to be the critical temperature for the change of mechanical properties, for 5 h to simulate the fire effect [8,11,17–19]. After cooling down, the connections were assembled. Bending tests were conducted on three end plate connections at room temperature and three connections where the bolts and plates were heated, simulating the fire effect, applied to column–beam connections. The post-fire behavior of the beam–column connections was monitored by measuring deformations with displacement transducers. Hereby the comparison of the behavior of connections with heat were compared with ambient temperature. At the end of the experiments, displacements in the beams were measured, and moment–rotation curves, failure modes, and the rigidity of connections were calculated. The experiments were also modeled in finite elements, and the finite element model will allow for the use of different geometries in future studies on connections. The presented information will assist designers in selecting the connection that will exhibit the best performance in cases of comparisons of end plate connections with different lengths and after fire loading.

2. Materials and Methods

2.1. Connection Details

The end plate details were designed according to Eurocode-3 (EC-3) [22,23]. To observe the behavior of the connections, the column cross-section was selected with a larger cross-section of HEB 180, because separation of the beams (IPE 140) from the column through the connection was desired. S235 steel grade and grade M8 bolts were chosen in all experiment details. The thickness of the end plate was kept constant and three different lengths of end plate connections were designed to examine the effect of the end plate length on behavior: smaller than the beam depth–partial end plate, equal to the beam web depth–flush end plate and larger than the beam web depth–extended end plate. The thickness of the plates was chosen as 10 mm, and the cross-sectional details of the end plates are shown in Figure 1.
In the study, 3 connections at ambient temperature and 3 post-fire connections were performed. The model denotation is “Model 1.R/T”. The first number next to the model indicates the type of connection, and the next number indicates the heat; thus, “R-model is at ambient temperature-reference”; “T-model is 600 °C temperature effect”. The experimental matrices performed in the study are shown in Table 1.

Table 1. Joint models in the study.

<table>
<thead>
<tr>
<th>No</th>
<th>Joint Type</th>
<th>Temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>20 °C</td>
</tr>
<tr>
<td>Model 1</td>
<td>Partial end plate</td>
<td>Model 1.R</td>
</tr>
<tr>
<td>Model 2</td>
<td>Flush end plate</td>
<td>Model 2.R</td>
</tr>
<tr>
<td>Model 3</td>
<td>Extended end plate</td>
<td>Model 3.R</td>
</tr>
</tbody>
</table>

2.2. Experimental Setup

Steel elements reach their limit deflection values at temperatures approximately between 510 °C and 650 °C under normal design loads, and they achieve their mechanical properties when cooled [8,13]. Therefore, in this study, end plates and bolts were exposed to high temperatures, which were increased to 600 °C with the standard curve. And then the components were heated at 600 °C for 5 h (Figure 2). After the connection elements were cooled, 1800 mm long columns were fixed in the frame, and 1860 mm long beams and connection elements were replaced in every experiment model. Three-point bending tests were performed (Figure 3). During the experiment, a load was applied vertically at the midpoint with a constant speed using a hydraulic loading device with a capacity of 1000 kN. The load was read using a load cell placed between the midpoint of the upper flange of the beam and the loading device (Figure 3). The similar experimental setup has been used in other studies as well [24–28]. Seven displacement transducers (LVDT) (P1–P7) with scales of 100 and 150 mm were placed on the bottom flange of the beam at the midpoint of the beam and at two equal points between the midpoint and the beam column connection (Figure 3). Thus, vertical displacements in the beam were measured. Data from the displacement transducers and load cell were recorded via a data logger during the experiment. In real time, the load-displacement curve was followed from the monitor in the computer. After reaching the maximum load-bearing capacity of the beam, the condition of the beam was kept under observation in the real time. The experiment was ended when the deformation or failure of the beam, or an increase in the deflection of the beam, occurred without the beam falling onto displacement transducers.
2.3. Moment–Rotation Relationships

The behavior of beam–column connections is determined by the moment resistance, rotational stiffness, and rotational capacity of the connection, which are drawn from the moment–rotation curve of the connection [29]. To determine the engineering behavior of the moment–rotation curve, it is necessary to transition from the load–deformation curve to the moment–rotation curve. In the scope of this study, the mathematical equations proposed by Coelho and friends [29] were used to obtain the moment–rotation curves of the connections.

The bending moment of the connection was calculated accordingly (1).

\[
M = P \times L_{\text{load}}
\]  

Here, \( M \) represents the bending moment of the connection, \( P \) represents the load applied to the beam, and \( L_{\text{load}} \) represents the distance from the applied load point to the connection.

\[
\theta = \arctan \left( \frac{- \frac{P}{EI} \left( \frac{X^3 DTi}{6} \right) - \left( \frac{L_{\text{load}}X^2 DTi}{2} \right)}{L} \right)
\]

The solution of Equation (2) gives the elastic rotation (\( \theta \)) of the beam. In Equation 2, \( E \) is the elastic modulus of the beam, \( I \) is the moment of inertia of the beam, and \( X \) is the distance between the LVDT and the connection.

As mentioned before, the moment–rotation curves inform about the moment resistance, rotational stiffness, and rotational capacity of the connection. In Eurocode-3, there are more details about the moment–rotation behavior of the connection, which is illustrated in the literature, as shown in Figure 4 [30].
Moment–rotation diagrams have some important points. $M_{Rd}$ is the plastic bending strength of the connection at the point where the initial stiffness line intersects with the post-yield stiffness line. $\theta_{M,Rd}$ is the rotation at this point. $\theta_{M,max}$ is the rotation corresponding to the maximum bending moment, $M_{j,max}$. The difference between the initial and post-yield stiffnesses of the connection is described as the knee range (KR). The moment range of this knee range region is between $M_{min,K,R}$ and $M_{up,K,R}$. The lower limit of rotation for this knee range region is $\theta_{min,K,R}$, and the upper limit is $\theta_{up,K,R}$. $M_{Cd}$ is the bending moment capacity of the connection. $\theta_{Cd}$ is the rotation value corresponding to this moment.

The length of the yield plateau on the moment–rotation curve represents the ductility of the connection. The ductility coefficient ($\Psi$) of the connection is calculated by Equation (3), depending on the plastic strength ($\theta_{M,Rd}$) and the total rotation capacity ($\theta_{Cd}$) of the connection [29].

$$\psi_j = \frac{\theta_{Cd}}{\theta_{M,Rd}} \tag{3}$$

The rotation values at maximum load and the corresponding ductility levels, $\Psi_{j,max,load}$, are calculated as follows in Equation (4) [29].

$$\psi_{j,max,load} = \frac{\theta_{M,j,max}}{\theta_{M,Rd}} \tag{4}$$

2.4. Finite Element Method

All the experiments were modeled using the finite element method in ANSYS Workbench R2 2020 [31]. SOLID 186 was used for the finite element model as a solid modeling technique. The structural framework of beams, columns, and plates was created by the Sweep Method (Figure 5). This method provided the homogeneity of the mesh elements.
Tensile tests were conducted on beam–column steels to ensure that the finite element model better reflects the experimental data. As no heat was applied to the columns and beams in the test matrices, the tensile tests were performed on coupons of beams and columns at ambient temperature. The mechanical properties of the other steels were obtained according to the specifications. In finite element analysis, the actual stress–strain curves obtained from the experiment for beam and column steel are multilinear, the mechanical properties of the plates are described as multilinear for S235 steel, and the mechanical properties for bolts are defined as bilinear for Grade 8.8 steel (Table 2).

**Table 2. Mechanical properties of the steel.**

<table>
<thead>
<tr>
<th>The Material</th>
<th>Yield (MPa)</th>
<th>Failure (MPa)</th>
<th>Moduls of Elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>End plates (S235)</td>
<td>235</td>
<td>360</td>
<td>200.000</td>
</tr>
<tr>
<td>Bolts (Grade 8.8)</td>
<td>640</td>
<td>800</td>
<td>200.000</td>
</tr>
<tr>
<td>Beam (S235)</td>
<td>251</td>
<td>391</td>
<td>200.000</td>
</tr>
<tr>
<td>Column (S235)</td>
<td>351</td>
<td>450</td>
<td>200.000</td>
</tr>
</tbody>
</table>

In EC-3, the degradation of mechanical properties such as elastic modulus, yield strength, and thermal properties such as thermal conductivity and specific heat with temperature is detailed. Therefore, reduction factors used to plot stress–strain curves of steel at elevated temperatures are shown in Table 3. The thermal properties of steel have been appropriately calculated according to Eurocode-3 for every different temperature level. The unit volume weight of steel is taken as 10 kg/m³, independent of temperature. In accordance with EC-3 [23], the values of specific heat, thermal conductivity and relative thermal elongation of steel are shown in Figures 6–8, respectively. At elevated temperatures, the material properties of beams, columns, and plates have been reduced according to the material reduction factors specified in Eurocode-3. The mechanical properties of bolts are also weakened by heat exposure, similar to plates. Studies have been conducted in the literature regarding the reduction factors to be used for bolts [32]. So, for the bolt, the reduction factor is taken as 0.22, based on Kirby’s study.

**Table 3. Reduction factors for stress–strain curves of steel at elevated temperature [23].**

<table>
<thead>
<tr>
<th>Steel temperature, (\theta_a) (°C)</th>
<th>Reduction Factors for Yield Stress (f_y), and Young’s Modulus (E_a), at Steel Temperature (\theta_a)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Reduction Factor (Relative to (f_y)) for Effective Yield Strength</td>
</tr>
<tr>
<td>20</td>
<td>1</td>
</tr>
<tr>
<td>100</td>
<td>1</td>
</tr>
<tr>
<td>200</td>
<td>1</td>
</tr>
<tr>
<td>300</td>
<td>1</td>
</tr>
<tr>
<td>400</td>
<td>1</td>
</tr>
<tr>
<td>500</td>
<td>0.78</td>
</tr>
<tr>
<td>600</td>
<td>0.47</td>
</tr>
</tbody>
</table>
According to Bursi and Jaspart, it is necessary to represent the real behavior for a numerical analysis to be close to realistic [33]. Once the stress–strain curve of the material is described at normal room temperature, stress–strain curves at different elevated temperatures can be easily specified according to the reduction factors in Eurocode-3 [34].

In the literature, experimental and numerical studies were presented to investigate the mechanical properties of steels during and after the fire [4,8–11,35–37]. Some of these studies state that steel recovered some of its mechanical properties after exposure to fire.
up to a certain temperature [37]. This specialty is important for the reuse of steel. However, the standards for reusing steel after a fire are limited. The British standard BS 5950-Part 8 informs about the use of steel after a fire [38]. The Chinese Standard CEC252:2009 represents the reduction factor for the yield strength of hot-rolled steel (Q235, Q345) after a fire [39]. However, this reduction factors cannot be used for other types of steel. In Eurocode-3, the standard about the reuse of steel has been canceled [37]. Therefore, a wide range of literature reviews have been carried out to create a numerical model of the post-fire behavior of steel.

Yang et al. [40] investigated the post-fire behavior of concrete-filled steel tubes by experimental and finite element methods. They conducted a four-phase analysis to determine the behavior of structures under fire and post-fire. The phases were named as the ambient temperature phase, heating phase, cooling phase, and post-fire phase.

While the temperature of the structure increases, additional deformations occur in the structure. Therefore, the mechanical properties of the material depend on the loading and temperature history. If a structure does not collapse when it reaches the maximum fire temperature, it may collapse during the cooling phase [41]. Real fires have many different temperature–time relationships in the real time. In the study, the ISO 834 fire curve was used to heat the bolts and plates (Figure 9). Yang et al. [40] and Hosseini et al. [41] added the cooling phase to the ISO 834 fire model to examine the effects of the cooling and post-fire phases [42].

![ISO 834 Standard fire curve and temperature in the study](image)

Figure 9. ISO 834 Standard fire curve and temperature in the study [42].

To evaluate a structure exposed to fire, the residual mechanical properties of the structural materials must first be assessed. In the heating stage, reduction factors provided by EC-3 were considered as mentioned before, while the following reduction factors were utilized to determine the post-fire properties [41]:

$$\frac{f_{yp}(T_{\text{max}})}{f_y} = \begin{cases} 1 & T_{\text{max}} \leq 400^\circ\text{C} \\ 1 + 2.33 \times 10^{-4}(T_{\text{max}} - 20) - 5.88 \times 10^{-7}(T_{\text{max}} - 20)^2 & T_{\text{max}} > 400^\circ\text{C} \end{cases}$$

(5)

in which, $f_y$ is the yield strength of steel at ambient temperature and $f_{yp}(T_{\text{max}})$ is the post-fire yield strength of the steel.
The solid modeling technique was used to achieve a high degree of correspondence between the analysis results and the experiment results. A rigid cylindrical part representing the load cell under the piston where the load is applied was modeled with the same diameter. In preparing a mathematical model to be fitted to the experiment, B, C, D, and E points represent the bottom and top ends of both columns designed as fixed supports. Point A represents the point where displacement is applied. The displacement value was obtained from the experiment, and then loading was made up to this displacement value along the +Y axis through the load cell. At the end, the problem was solved (Figure 10).

![Figure 10. Boundary conditions.](image)

The pre-tension force was applied to the bolts before displacement was applied. The pre-tension force in the bolts was calculated using the formula [43]

\[ F_0 = n \times s_y \times A_s \]  

(6)

Here, \( F_0 \) is the bolt pre-tension force, \( n \) is a coefficient between 0.5 and 0.7, \( s_y \) is the bolt yield stress, and \( A_s \) is the bolt cross-sectional area.

3. Experimental Results

In the experiments at the ambient temperature, beams and columns were compounded at room temperature and the frame was loaded until the beam collapsed. In the post-fire experiments, the connection parts were heated in the furnace at 600 °C for 5 h. After cooling, the beams and columns were compounded at room temperature, and the frame beam was loaded. The experiment has been ended and concluded due to occurrences such as bolt failure, excessive beam deflection, and significant deformation of the end plate. The obtained experimental data were compared within the connection models and heat variables. The maximum load that Model 1.R could carry was 81 kN. When the connection parts were heated, the maximum load on the beam decreased by 4% to 77.8 kN, and the maximum displacement value increased by 9% (Figure 11).

The end plate was equal to the beam depth in Model 2.R, and the experiment was terminated due to observed bolt failures. The maximum load was 101.8 kN. When the connection elements were heated, the maximum load decreased by 23.2% to 78.2 kN. Additionally, the maximum displacement was 40.6 mm at room temperature and 31.6 mm post-fire. Heating the connection elements reduced the load-bearing capacity of the beam. The displacement in Model 2.T was less than that in Model 2.R because of the significant decrease in load-bearing capacity caused by the heat. When analyzing Figure 8, it can be seen that the heat-influenced model had 23.2% less load with 17.6% more vertical displacement compared to the heat unaffected model. This indicates that the heat reduced the rigidity of Model 2.

In Model 3, where the end plate is larger than the beam depth, the experiments were terminated due to observed bolt failures. At the normal room temperature, the beam could have a load of 125.8 kN, but when the connection elements were heated, the load...
decreased by 22.5% to 97.5 kN. While a maximum vertical displacement of 32 mm occurred in Model 3.R, this value increased by 64.4% to 52.6 mm in Model 3.T. The heat reduced the load-bearing capacity of the beam in a connection where the end plate is larger than the beam depth and increased the amount of displacement happening at the midpoint of the beam. When examining the load–displacement graph, it can be observed that the connection without heat reached the maximum load at 11.6 mm displacement, while the connection with heated elements reached 23.1 mm (Figure 8).

![Graphs showing load-displacement for different models](image)

**Figure 11.** Heat effects on models.

A comparison was made between Model 1, Model 2, and Model 3, which have different end plate lengths, both among themselves and under the effects of heat (Figure 12). In the models under an ambient temperature, the largest maximum load-bearing capacity was 125.8 kN in Model 3.R, followed by 101.8 kN in Model 2.R, and the smallest was 81 kN in Model 1.R. Thus, increasing the length of the end plate from 80 mm to 165 mm increased the beam’s maximum load by 25.7%. Increasing the length of the end plate from 80 mm to 230 mm increased the beam’s maximum load by 55.3%. It is clear that increasing the length of the end plate has increased the beam’s load-bearing capacity. When investigating the vertical displacement values at the maximum load in the graphs in Figure 9, it can be observed that Model 1.R approached 12.4 mm at 81 kN, Model 2.R approached 11.4 mm at 101.8 kN, and Model 3.R reached 11.6 mm at 125.8 kN. Although the difference is big in the maximum loads of the beams, the connections achieved the same displacement, nearly reaching the maximum load values. This situation indicates that the increase in end plate length plays a significant role in the increase in the connection’s rigidity. The increase in end plate length allowed the beams to be loaded to bigger loads in the experiments. When Model 1.R reached 78% of its load bearing capacity, when Model 2.R reached 70% of its load bearing capacity, and when Model 3.R reached 60% of its load bearing capacity, the loadings were stopped in the experiment. It should be taken into consideration that to reach this load value, Model 1.R with the shortest end plate depth, had more displacement. In other words, the shorter end plate depth decreased the connection rigidity. The increase in the end plate depth also reduced the permanent deflection value in the beam.

The result of the maximum load is similar in the post-fire behavior (Figure 12). Here-under, the maximum load has been enhanced with the increase in the length of the end plate. Model 3.T was able to resist 97.5 kN, which is 24.7% more than Model 2.T and 25.3% more than Model 1.T. The deflection amounts of the beams at these maximum loads have increased with the increase in the length of the end plate; Model 1.T reached the maximum
deformation with a deformation of 17.6 mm, Model 2.T reached 13.4 mm, and Model 3.T reached 23.1 mm. The increase in the length of the end plate increased the maximum deformation of the beam in the post-fire behavior. At the end of the experiment with Model 3.T, a deformation of 52.6 mm occurred, which is 66.5% more than Model 2.T and 11.5% more than Model 1.T. It has been determined that the increase in end plate length increases the permanent deformation in the post-fire behavior.

![Figure 12](image.png)

**Figure 12.** Effect of 20 °C and 600 °C temperatures on the end plate length.

The maximum load-carrying capacities of all of the end plate models are compared in Table 4. According to this, the heat affected Model 2.T the at least, which has the shortest connection with an 80 mm end plate. Although the decrease percentages of Model 2.T and Model 3.T are approximate, considering the maximum loads, it is thought that in the post-fire behavior, the model with the highest load-bearing capacity is Model 3.T with an end plate larger than the beam depth.

<table>
<thead>
<tr>
<th>Model</th>
<th>20 °C Max. Load (kN)</th>
<th>600 °C Max. Load (kN)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>81</td>
<td>77.8</td>
<td>4</td>
</tr>
<tr>
<td>Model 2</td>
<td>101.8</td>
<td>78.2</td>
<td>23.2</td>
</tr>
<tr>
<td>Model 3</td>
<td>125.8</td>
<td>97.5</td>
<td>22.5</td>
</tr>
</tbody>
</table>

4. Moment–Rotation Relationships Obtained from Experiments

The moment–rotation curves were calculated from the load–vertical displacement curves of the experiments. Every moment–rotation curve gives understanding about the stiffness, strength, and rotation values of the connections.

The initial stiffness and the post-yield stiffness (knee range) of the connection increased with the length of the end plate at the ambient temperature. When Model 3.R was compared to Model 2.R and Model 1.R, it was seen that the knee range distance increased by 10.22% and 59.61%, respectively. Heat reduced the knee range. Additionally, when examining the distance differences at the heat-affected connections, it was considered that these differences increased with the lengthening of the end plate (Table 5).

The plastic bending resistances (M_{pl}) in the same models with and without heat influence increased with the lengthening of the end plate. Heat reduced the plastic bending resistance of the connection. The resistance of Model 1.T decreased by 3.06% compared to Model 1.R, the resistance of Model 2.T decreased by 17.75% compared to Model 2.R, and the resistance of Model 3.T decreased by 9.53% compared to Model 3.R.

In the models at an ambient temperature, the maximum bending moment occurred as 121.41 kN.m in Model 3.R, which had an end plate larger than the beam depth. In the models where the connection was heated, the maximum bending moment was 94.05 kNm in Model 3.T, which also had an end plate larger than the beam depth. It can be observed that the increase in the length of the end plate increased the maximum bending moment.
of the connection. Heating reduced the bending moment of the connection by 22.54% in the same model.

In the models at an ambient temperature, an increase in the length of the end plate decreased the plastic bending resistance rotation \( (\theta_{Mj,Rd}) \) values of the connection. The plastic bending resistance rotation in Model 3.R was 5.1% less than in Model 2.R and 46.97% less than in Model 1.R. Heating the connection parts also resulted in an increase in the rotation values. In the heat-affected models, the rotation values in Models 1, 2, and 3 increased by 17.80%, 45.58%, and 91.38%, respectively, compared to room temperature models. The increase in the length of the end plate and the heating increased the difference in the decrease in rotations.

The rotation values at the maximum bending moment \( (\theta_{Mj,max}) \) increased with heat influence in all models except Model 2. This indicates that the connection softened with heat, and these models reached the maximum moment value with more vertical displacement compared to without-heat models. In the room temperature models, an increase in the length of the end plate provided less rotation at the maximum moment value. The shortest bearing plate length, Model 1.R, had a rotation \( (\theta_{Mj,max}) \) of 24.91% less than Model 3.R. The rotation capacity of the connection \( (\theta_{Rj}) \) decreased with the increase in the length of the end plate in the ambient temperature models (Table 5).

The increase in the end plate height expelled the ductility coefficient \( (\nu) \) of the connection. The situation seems different in Model 2. As mentioned before, the total rotation related to the ductility coefficient and the total rotation changed according to the end of the test. Model 3.R gained 43.45% compared to Model 1.R, and Model 2.R gained 75.34% compared to Model 1.R. The results are similar in the heat-affected experiments—the ductility coefficients decreased. For Models 1, 2, and 3, the ductility coefficients decreased by 7.43%, 48.33%, and 12%, respectively (Table 6).

The maximum ductility coefficient means the ratio of the maximum rotation value of the connection to the plastic rotation resistance value. The increase in the length of the end plate has enhanced this coefficient. Specifically, the maximum ductility coefficient of Model 1.R increased by 88.69% from 1.68 to 3.17 in Model 2.R, and in Model 3.R, it increased by 51.19% to reach a value of 2.54. Similar results were observed in the heat-affected results.

The maximum ductility coefficient is figured according to the rotation value corresponding to the connection’s maximum moment, while the ductility coefficient is figured according to the total rotation value. For this reason, if the connection’s ductility coefficient is bigger than the maximum ductility coefficient, it means that the connection continued to carry the load by rotating without an increase in the load after reaching the maximum load.
Table 5. Moment–rotation characteristic values.

<table>
<thead>
<tr>
<th>Exp. Group</th>
<th>Exp. No</th>
<th>KR (Knee–Range)</th>
<th>Resistance (kN m)</th>
<th>Stiffness (kN m/mrad)</th>
<th>Rotation (mrad)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$M_{j,Rd}$</td>
<td>$M_{j,\text{max}}$</td>
<td>$M_{\text{cd}}$</td>
</tr>
<tr>
<td>20 °C Exp.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Model 1.R</td>
<td>27.59–75.8</td>
<td>71.84</td>
<td>78.57</td>
<td>59.16</td>
<td>8.84</td>
</tr>
<tr>
<td>Model 2.R</td>
<td>25.32–95.13</td>
<td>83.12</td>
<td>98.21</td>
<td>70.69</td>
<td>18.06</td>
</tr>
<tr>
<td>Model 3.R</td>
<td>28.01–104.96</td>
<td>90.58</td>
<td>121.41</td>
<td>72.24</td>
<td>21.04</td>
</tr>
<tr>
<td>600 °C Exp.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Model 1.T</td>
<td>40.69–72.34</td>
<td>69.64</td>
<td>75.04</td>
<td>61.02</td>
<td>7.51</td>
</tr>
<tr>
<td>Model 3.T</td>
<td>60.69–87.53</td>
<td>81.95</td>
<td>94.05</td>
<td>52.93</td>
<td>9.97</td>
</tr>
</tbody>
</table>
Table 6. The ductility values.

<table>
<thead>
<tr>
<th>Exp. Group</th>
<th>Exp. No</th>
<th>Rotation (mrad)</th>
<th>( \psi )</th>
<th>( \psi_{\text{max load}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( \theta_{\text{M,Rd}} )</td>
<td>( \theta_{\text{M,\text{max}}} )</td>
<td>( \theta_{\text{C,d}} )</td>
</tr>
<tr>
<td>20 °C Exp.</td>
<td>Model 1.R</td>
<td>8.09</td>
<td>13.59</td>
<td>41.31</td>
</tr>
<tr>
<td></td>
<td>Model 2.R</td>
<td>4.52</td>
<td>14.31</td>
<td>40.51</td>
</tr>
<tr>
<td></td>
<td>Model 3.R</td>
<td>4.29</td>
<td>10.88</td>
<td>31.44</td>
</tr>
<tr>
<td>600 °C Exp.</td>
<td>Model 1.T</td>
<td>9.53</td>
<td>16.62</td>
<td>45.06</td>
</tr>
<tr>
<td></td>
<td>Model 2.T</td>
<td>6.58</td>
<td>11.49</td>
<td>30.47</td>
</tr>
<tr>
<td></td>
<td>Model 3.T</td>
<td>8.21</td>
<td>22.36</td>
<td>52.93</td>
</tr>
</tbody>
</table>

5. Finite Element Analysis Results

The load–displacement curves in the experiments were compared with the load–displacement curves from the numerical analysis (Figure 13). In all numerical analysis, similar vertical displacement was obtained in the similar maximum load values (Table 7).

![Comparison of load–displacement curves of finite elements and experiments.](image)

Table 7. Comparison of maximum loads in the finite elements and experiments.

<table>
<thead>
<tr>
<th>Model</th>
<th>Exp. Load (kN)</th>
<th>FEA Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1.R</td>
<td>81</td>
<td>82.84</td>
</tr>
<tr>
<td>Model 2.R</td>
<td>101.8</td>
<td>102.9</td>
</tr>
<tr>
<td>Model 3.R</td>
<td>125.8</td>
<td>127.9</td>
</tr>
<tr>
<td>Model 1.T</td>
<td>77.8</td>
<td>80.95</td>
</tr>
<tr>
<td>Model 2.T</td>
<td>78.2</td>
<td>79.66</td>
</tr>
<tr>
<td>Model 3.T</td>
<td>97.5</td>
<td>100.36</td>
</tr>
</tbody>
</table>

The equivalent stress distributions (von Mises) for all models were individually figured (Figure 14). Due to the increase in the mechanical properties of the heat, the equivalent stress values decreased (Table 8). The load-bearing capacity increased based on the increasing length of the end plate. So, stress at the beam also increased. It was considered that stresses increased in the beam midpoint in all models.
Figure 14. Examination of equivalent stress distributions (Model 1.R).

Table 8. Equivalent stress distributions revealed in numerical analysis.

<table>
<thead>
<tr>
<th>Model</th>
<th>Equivalent Stress Values (MPa)</th>
<th>Frame</th>
<th>End Plate</th>
<th>Beam Mid Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1.R</td>
<td>320.7</td>
<td>302.22</td>
<td>287.47</td>
<td></td>
</tr>
<tr>
<td>Model 2.R</td>
<td>298.34</td>
<td>253.04</td>
<td>287.82</td>
<td></td>
</tr>
<tr>
<td>Model 3.R</td>
<td>293.28</td>
<td>291.89</td>
<td>293.28</td>
<td></td>
</tr>
<tr>
<td>Model 1.T</td>
<td>290.41</td>
<td>183.29</td>
<td>282.4</td>
<td></td>
</tr>
<tr>
<td>Model 2.T</td>
<td>288.51</td>
<td>173.46</td>
<td>288.49</td>
<td></td>
</tr>
<tr>
<td>Model 3.T</td>
<td>310.09</td>
<td>182.87</td>
<td>310.02</td>
<td></td>
</tr>
</tbody>
</table>

After the experiment ended, the values of separation of the flanges from the column were measured, and photographs were taken. The experiment details and the details of the final step of numerical analyses were compared. It was marked that the separation values and the types of flanges were similar (Table 9).

Table 9. Comparisons of deformation and separation in joints.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td><img src="image" alt="5 mm / 3.9 mm" /></td>
<td><img src="image" alt="6.5 mm / 5.8 mm" /></td>
</tr>
</tbody>
</table>
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In the numerical analysis results, the bolt in the upper row where the maximum stress occurred was examined. The failure situation of the bolt was considered by examining the force and stress values of the same bolt. The failure conditions of the bolts were compared in the analysis and experiment, and it was observed that the behaviors were similar (Table 10). No failure was observed in the bolts, only in the analysis of the smallest end plate. Some of the bolts that failed at the end of the experiment are shown in Figure 15.

Table 10. Bolts examined in numerical analysis and experiments.

<table>
<thead>
<tr>
<th>Model</th>
<th>Numerical Analysis Stress (MPa)</th>
<th>Failure Condition</th>
<th>Experiment Failure Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1.R</td>
<td>791.57</td>
<td>No Failure</td>
<td>No Failure</td>
</tr>
<tr>
<td>Model 2.R</td>
<td>980.97</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>Model 3.R</td>
<td>952.45</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>Model 1.T</td>
<td>458.2</td>
<td>No Failure</td>
<td>No Failure</td>
</tr>
<tr>
<td>Model 2.T</td>
<td>606.94</td>
<td>Failed</td>
<td>Failed</td>
</tr>
<tr>
<td>Model 3.T</td>
<td>606.55</td>
<td>Failed</td>
<td>Failed</td>
</tr>
</tbody>
</table>

Figure 15. Some of the failed bolts.

The end plates started to deform at lower stress values depending on the heat. Deformation did not occur in the end plate until the stress reached a value of 165 MPa at room temperature in the model where the end plate is smaller than the beam depth. However, when the connection parts were heated, deformation started in the end plate under a stress of 56.15 MPa. In all models, the end plates began to deform at lower stress values because of the heat (Figure 16).

The unit deformation remained stable at 0.0001 mm/mm until reaching a yield stress of 235 MPa for the steel at room temperature. However, an increase in unit deformation was observed until reaching the same yield stress. The unit deformation value reached a value of 0.0003 mm/mm in the model where the end plate is equal to the beam depth and
a value of 0.0017 mm/mm where the end plate is bigger than the beam depth. In this case, the increase in plate length positively affected elastic behavior.

Although increasing the end plate length led to more unit deformation to reach the steel yield value in room temperature analysis, the analysis results showed the opposite. At the end of the analysis, a unit deformation of 0.0316 mm/mm was observed in the smallest end plate under a stress below 302.22 MPa. The end plate which is equal to the beam depth underwent a unit deformation of 0.0081 mm/mm under a stress of 253.04 MPa. The end plate which is bigger than the beam depth underwent a unit deformation of 0.0218 mm/mm under a stress of 291.89 MPa. At the end of the analysis, when the unit deformation and stress values in the end plates were studied, it was considered that the largest unit deformation occurred in the smallest end plate.

At the end of the analysis, the stress in the beam was decreased by 7% by increasing the end plate length, and the deformation in the beams was reduced by up to 45.75%. Thus, the reduction in the size of the end plate increased the deformation in the beam under similar stresses.

The end plate experienced more unit deformation under the effect of the heat at lower stresses compared to the beam. After reaching the maximum stress with heat, the end plate continued to deform under the same stress until the analysis ended—this is the general behavior. For example, in Model 1.T, the end plate reached the maximum stress at a unit deformation of 0.020 mm/mm and then continued to deform under the same stress up to a value of 0.051 mm/mm (Figure 16).

Heat reduced the mechanical properties of the end plates. So, the end plates contributed lower stresses and caused more unit deformation.

Figure 16. The variation in stress–strain curves for maximum stress points in plates and beams with temperature changes.

6. Conclusions

The goal of this study is to execute the post-fire behavior of different connections with varying end plate lengths. Steel elements regain their mechanical properties after cooling below temperatures of 600 °C. For this reason, 600 °C is chosen as the limit temperature threshold. In the first stage, experimental studies on steel beam–column connections at room temperature were conducted. In the second stage, to investigate the variable of heat, connection elements were exposed to 600 °C for 5 h to investigate the effect of heat, and flexural tests were conducted. In the third stage, moment–rotation relationships from the experiments were drawn and interpreted. In the fourth stage, finite element
analysis was performed, and these results were compared with experimental results. The results of the study can be summarized as follows:

1. Heating the connection elements reduces the load-bearing capacity of the connection while increasing the deflection in the beam.
2. Heating the connection elements reduced the connection stiffness.
3. Increasing the end plate length increased the load-bearing capacity of the beam and reduced the amount of deflection at room temperature. When the connection elements were heated, the increase in the end plate length increased both the load-bearing capacity and the value of deflection.
4. The plastic bending strength and the bending moment capacity of the connection increased with the increase in end plate length. However, heat reduced the plastic bending strength of the connection. The increase in end plate length increased the rotation capacity of the connection with heat.
5. Heat softened the connection. In other words, connections with heat reached the maximum moment value by performing more vertical displacement compared to without-heat models. In room temperature connections, the connection reached the maximum moment value with less rotation because of the increase in end plate length.
6. The ductility coefficient of the connection increased with the increase in end plate height in the room temperature connection. The ductility coefficient of the connection decreased with the heat.
7. Load-bearing capacity and the slope of the moment–rotation curve reduced because of the heat. For this reason, plastic bending strengths decreased with heat effects, too.
8. Heating and then reusing the connection elements reduced the load-bearing capacity of the beam. Even at the limit temperature threshold of 600 °C, the decrease in load-bearing capacity indicates that higher temperatures occurring in a fire could result in a more significant loss of load-bearing capacity. Therefore, even during the post-fire repair stage, collapses could occur.
9. The finite element model created in the numerical analysis, validated by the experimental results, can be used for future research, and the number of samples can be increased.
10. Frames with different material and connection details can be loaded up to a certain capacity and then exposed to fire, allowing for comparison between fire and environmental loads.
11. In the frame, seismic load can be applied first, and then fire effect can be studied.
12. In the frame, the post-fire seismic behavior of different joints can be studied via finite elements.

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