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# Behavior of bolted top-seat angle connections in fire

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ABSTRACT

Beam-to-column connections have been found to be of great significance in influencing structural behavior at ambient and elevated temperatures. When steel-framed structures are subjected to fire, the load bearing capacity is decreased and the behavior of the joints is of particular concern. Observations from full-scale fire tests and damaged structures confirm that connections have a considerable effect on the stability time of structural components in fire. The cost of high temperature tests on the broad range of connections used in practice means that their influence is not well detailed in current design codes. The paucity of data also limits the effective use of numerical models developed to simulate the behavior of complete structures at elevated temperatures. In this study, 12 full-scale tests were conducted at elevated temperatures on two types of bolted angle beam-to-column connections in order to investigate their resistance to fire. The failure modes and deformation patterns of these specimens were studied and the results are shown as rotation–temperature curves. In addition, the influence of different parameters such as thickness of the angles, the grade of bolts, and other geometrical and mechanical characteristics of the connections were studied.



## 1. Introduction

Structural steel frames consist of beams and columns connected together. The structural performance of the complete frame is affected by the behavior of the joints, which should be considered in any global analysis of the structure.

In conventional analysis and design of steel and composite frames, beam-to-column connections are assumed to behave either as 'pinned' or as fully 'rigid' joints [1]. Although the pinned or rigid assumption significantly simplifies analysis and design procedures, in practice, the actual joint behavior exhibits characteristics from a wide spectrum between these two extremes.

The difference between the two simplified joint types is that pin joints have rotational stiffness while rigid joints display flexibility. Designers may choose a more accurate representation of joint behavior for analysis and design, but many adopt simplified economical methods. Although these simplified approaches are sufficient for designs at ambient temperatures, when steel-framed structures are subjected to fire the behavior of the joints within a frame exerts an even greater influence on overall response. If the behavior of these connections is not considered properly, the analysis may misrepresent the performance of a structure during a fire.

As steel structures are widely used in buildings and are sensitive to fire, researches have been focused on studying the influence of high temperatures on the behavior of steel structures.

The traditional method for fire protection is to use fire-resistant coatings, which increase the cost and time of construction [2]. To minimize the need for coatings, extensive experimental and analytical studies of the effects of temperature on structures seem to be necessary. Tests can be conducted either on individual members or the whole structure.

For this purpose, some tests have been focused on the influence of temperature on the response of structural connections. The tests conducted by CTICM [3] in 1976 investigated the performance of high strength bolts at elevated temperatures. British Steel [4] also carried out two tests in 1982 on a moment-resisting connection.

Experimental studies were also conducted by Lawson [5] and Leston-Jones [6], however, these tests only provided useful data for a limited range of conditions using relatively small section sizes.

In 1999, Al Jabari et al. [7] Conducted 20 experimental tests on five types of structural connections at elevated temperatures. In 2006, Wei-Yunj et al. [8] conducted a full-scale test on four connections at elevated temperatures. In addition to the studies conducted on connections, other experimental tests have been conducted to specifically study the behavior of bolts, which are a major component of structural connections. Kirby [9] conducted a series of experimental tests in 1995 to determine the strength degradation of grade 8.8 bolts in a temperature range from 300

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**Table 1**
Summary of experimental tests conducted on connections

| Reference | Connection type | Beam size | Column size | Bolt size & number | End-plate thickness (mm) | Orientation & arrangement |
|---|---|---|---|---|---|---|
| Lawson [5] | (2) extended bare | 305 × 165 × 40 UB (S275) | 203 × 203 × 52 UC (S275) | 6 M20 | 12 | Major beam–column |
| | (2) flush bare | 305 × 165 × 40 UB (S275) | 203 × 203 × 52 UC (S275) | 6 M20 | 12 | Major beam–column |
| | (1) web cleat bare | 305 × 165 × 40 UB (S275) | 203 × 203 × 52 UC (S275) | 6 M20 | 12 | Major beam–column |
| | (1) flush bare | 305 × 165 × 40 UB (S275) | 203 × 203 × 52 UC (S275) | 6 M20 | 12 | Major beam–column |
| | (1) flush bare | 305 × 165 × 40 UB (S275) | | 6 M20 | 12 | Major beam–column |
| | (1) web cleat bare | 305 × 165 × 40 UB (S275) | | 6 M20 | 12 | Major beam–column |
| | (1) flush composite | 305 × 165 × 40 UB (S275) | 203 × 203 × 52 UC (S275) | 6 M20 | 12 | Major beam–column |
| | (1) web cleat composite | 305 × 165 × 40 UB (S275) | 203 × 203 × 52 UC (S275) | 6 M20 | 12 | Major beam–column |
| | (1) flush composite | 305 × 165 × 40 UB (S275) | | 6 M20 | 12 | Major beam–column |
| Leston-Jones [6] | (5) flush bare | 245 × 102 × 22 UB (S275) | 152 × 152 × 23 UC (S275) | 6 M16 | 12 | Major beam–column |
| | (5) flush composite | 245 × 102 × 22 UB (S275) | 152 × 152 × 23 UC (S275) | 6 M16 | 12 | Major beam–column |
| Aljabri [7] | (4) flush bare | 254 × 102 UB (22) | 152 × 152 UC (23) | 6 M16 | 8 | Major beam–column |
| | (4) flush bare | 356 × 171 UB (51) | 254 × 254 UC (89) | 8 M20 | 10 | Major beam–column |
| | (3) flexible bare | 356 × 171 UB (51) | 254 × 254 UC (89) | 8 M20 | 8 | Major beam–column |
| | (5) flexible composite | 356 × 171 UB (51) | 254 × 254 UC (89) | 8 M20 | 8 | Major beam–column |
| | (4) flexible composite | 610 × 229 UB (101) | 305 × 305 UC (137) | 14 M20 | 10 | Major beam–column |
| Lou and Li [15] | (2) extended end plate | H300 ∗ 160 | H240 ∗ 240 | 8 M20 | 10 | Major beam–column |
| Wei-yong wang [8] | (1) flush | H 250 × 125 × 6 × 9 | H 244 × 175 × 7 × 11 | 8 M20 | 12 | Major beam–column |
| | (1) flush | H 250 × 125 × 6 × 9 | H 244 × 175 × 7 × 11 | 8 M20 | 12 | Major beam–column |
| | (1) flush | H 250 × 125 × 6 × 9 | H 244 × 175 × 7 × 11 | 8 M20 | 16 | Major beam–column |
| | (1) flush | H 250 × 125 × 6 × 9 | H 244 × 175 × 7 × 11 | 8 M20 | 16 | Major beam–column |

to 700 °C. The load capacity of bolts decreases significantly with temperature, as reported. On the basis of these results, several tri-linear equations were proposed for the SRF (Strength Reduction Factor). Theodroo [10] in 2001 also investigated the behavior of bolts at elevated temperatures. The results of this study showed that the SRF equations proposed by Kirby are valid and the SRF equations presented in EC3 [11] and BS5950 [12] are also applicable. Several other tests have also been carried out on steel members to study the influence of elevated temperatures [13,14].

A summary of experimental tests of structural connection strength conducted at elevated temperatures is shown in Table 1.

To further investigate the behavior of connections at elevated temperatures, twelve full-scale tests were conducted on two types of bolted angle connections for this study. The results are presented as rotation–temperature curves for various configurations. In consideration of the preceding studies on the behavior of connections at elevated temperatures, several factors were taken into account to improve the results of this current study:

1. In all of the tests, full-scale specimens were made and tested.
2. In some studies carried out in the past where there were large rotations in the connections, the beam struck the frame of the test furnace and made it impossible to continue the test [7]. In this study, the furnace was specially designed to prevent this problem from reoccurring.
3. Since the only purpose of this study was to examine the behavior of connections at elevated temperatures, the sections of beam and column in the furnace were covered by a 2.5 cm ceramic fiber cover. Only the connection area consisting of the column flange and the web flange in the vicinity of the connection were exposed to the fire. This method significantly reduced temperature increases in these sections.
4. To increase the accuracy of the tests, the method of increasing the temperature was close to the methods mentioned in current temperature codes.

In this study, in addition to studying the effects of fire on angle connections, the behavior of bolts and their failure modes at elevated temperatures were also studied and compared to the preceding studies. The configuration of the tests was selected in a way that made it possible to study the effect of other parameters such as: bolt grade, angle thickness, presence or absence of web angles, and the amount of moment applied to the connection. As mentioned before, previous studies have been carried out to investigate the behavior of connections at elevated temperatures, but since there are no studies in which the effect of elevated temperatures on top and seat angle connections have been thoroughly examined, it was essential to carry out the current investigation.

## 2. Experimental tests

### 2.1. Arrangement and instrumentation of the tests

The tests were conducted in a gas-fired furnace that was designed for this study. The test setup is shown in Fig. 1.

The temperature of the furnace was increased according to ISO834 and ASTME119 [16,17] curves as shown in Fig. 2.

The instrumentation included clinometers for measuring rotation, displacement transducers (LVDT), load cells and thermocouples. Both analogue and digital output devices were monitored through a TDS-303 data logger made by the Japanese TML Company. Full details about the instrumentation and test procedures have been previously published [18].

The test procedure consisted of three steps. First, the specimens were loaded to reach a predetermined load level. Then the fire was started in the furnace while a constant load was applied to the specimens. Finally, when the connection failure occurred, the fire was extinguished in the furnace. Images were captured during the tests at different temperatures and processed using image processing software to produce a temperature–displacement plot.

### 2.2. Specimen details

All the specimens were configured as a symmetric cruciform arrangement consisting of a single 80 cm high column of IPE300 section connected to two 240 cm long cantilever beams of IPE 220. The load was applied on a point of each beam two meters from the end of the beam. All of the bolts in each specimen were tightened to 150 N m using a torque wrench to ensure consistency.

The cruciform test arrangement was selected because: (a) it requires a less expensive test rig than the corresponding cantilever arrangement, (b) it provides an indication of the variability of

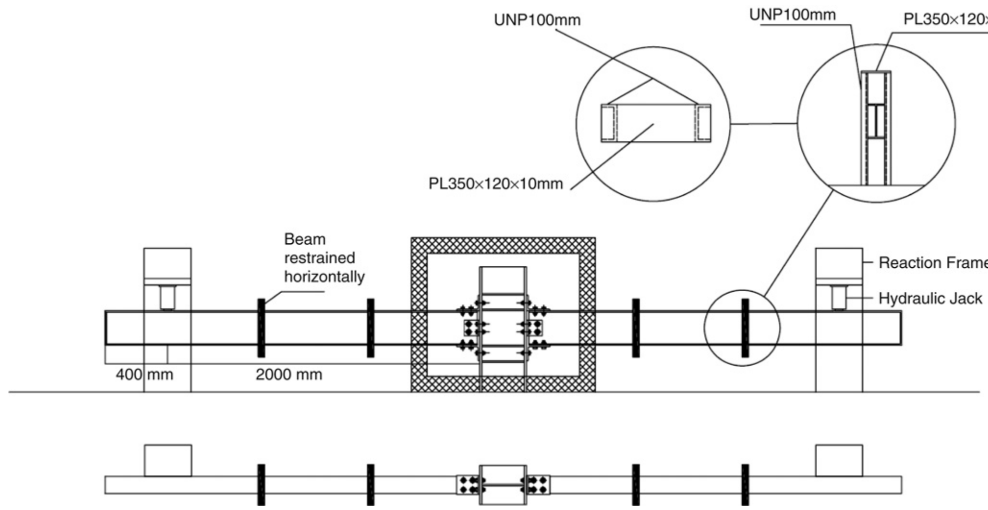



**Fig. 1.** Test set up.

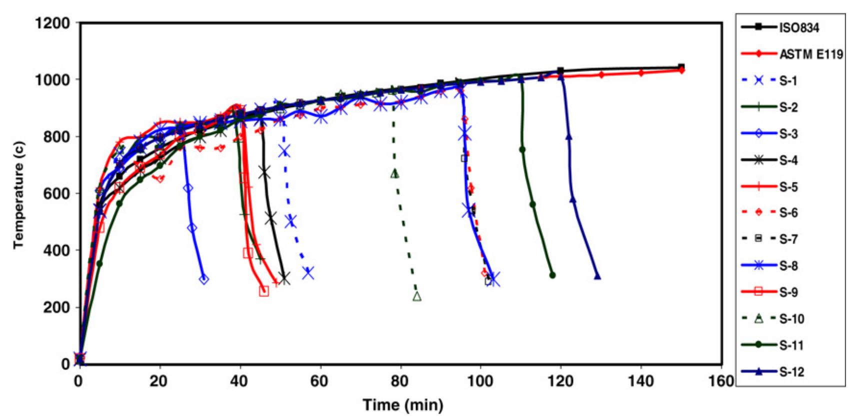



**Fig. 2.** Recorded furnace temperatures, ISO and ASTM standard fire temperatures.

**Table 2**
Details of specimens

| Specimen number | Group number | Angle size (mm) | Gap (mm) | Grade of bolt | No of nuts |
|---|---|---|---|---|---|
| 1 | 2 | 150 ∗ 100 ∗ 15 | 0 | 8.8 | 1 |
| 2 | 1 | 150 ∗ 100 ∗ 15 | 0 | 8.8 | 1 |
| 3 | 1 | 100 ∗ 100 ∗ 10 | 15 | 8.8 | 1 |
| 4 | 1 | 100 ∗ 100 ∗ 15 | 15 | 8.8 | 1 |
| 5 | 2 | 150 ∗ 100 ∗ 15 | 15 | 8.8 | 1 |
| 6 | 1 | 100 ∗ 100 ∗ 15 | 15 | 10.9 | 1 |
| 7 | 2 | 150 ∗ 100 ∗ 15 | 15 | 10.9 | 3 |
| 8 | 1 | 100 ∗ 100 ∗ 10 | 15 | 10.9 | 1 |
| 9 | 1 | 150 ∗ 100 ∗ 15 | 15 | 8.8 | 1 |
| 10 | 1 | 100 ∗ 100 ∗ 15 | 15 | 10.9 | 3 |
| 11 | 2 | 150 ∗ 100 ∗ 15 | 15 | 10.9 | 1 |
| 12 | 2 | 150 ∗ 100 ∗ 15 | 15 | 10.9 | 1 |

Gap: Distance between column and beam.

the nominally identical connections on either side of the column, and (c) two connections (east-west) are tested in every specimen. Lateral bracing was provided to prevent beam torsion during the test. These braces were designed according to the maximum torsional moment of the beam. The braces and their orientation are shown in Fig. 1. In total, 12 experimental tests were conducted on two different connection types.

Connection Type I: (Specimen without Web Angle) (SOW)

Connection Type II: (Specimen with Web Angle) (SWW)

Connection Type I (SOW) consists of two angles, one connected to the top flange of the beam and the other connected to the bottom flange of the beam. The total system is then bolted to the flange of the column. Each angle is bolted to the flange of the beam by six M16 bolts and to the flange of column by two M16 bolts. The details of this type of connection are shown in Fig. 3.

Connection Type II (SWW) has two additional angles beyond what is used for Connection Type I. These angles are bolted to the web of the beam on one side and to the flange of the column on the other side. Web angles are connected to the web of the beam by two M16 bolts and to the flange of the column by two M16 bolts. Details about this group of connections are shown in Fig. 4. Details of all specimens are given in Table 2.

### 2.2.1. Specimen loading

Since the force applied to the connections in this study is a combination of the moment due to the force of the hydraulic jacks and the temperature forces due to the temperature increase, it is obvious that the higher the applied moment, the lower the connection failure temperature will be. Since the scope of this study is to investigate the behavior of connections at elevated temperatures, the moments were selected in a manner such that premature connection failure would not occur and the connection can tolerate higher temperatures. In this manner, the behavior of connections can be studied at higher temperatures. Since the effects of several parameters were examined in this study, it was necessary for the moment force to be equal for all connections. In this manner, it was possible to compare the effects of variable parameters in two different tests. For this reason, the rotational capacity of each connection was calculated using classic methods and then that moment was applied to each connection through a

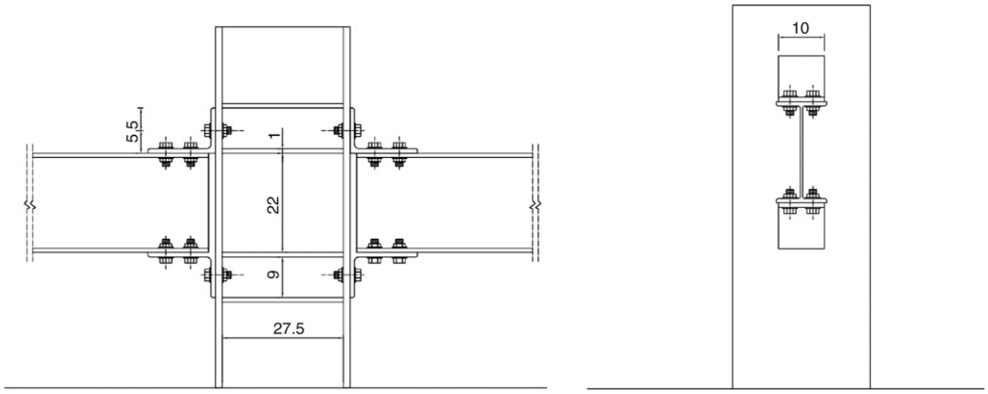



**Fig. 3.** SOW Connection detail.

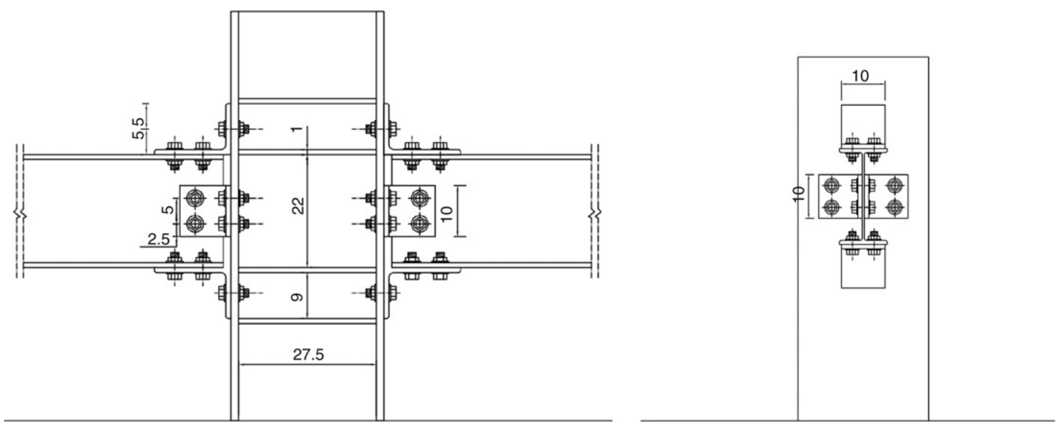



**Fig. 4.** SOW connection detail.

**Table 3**
Level of loading for connection tests

| Specimen number | Group number | Moment ($M$) level | Applied $M$ (kN m) | Average recorded $M$ (kN m) |
|---|---|---|---|---|
| 1 | 2 | .4 ∗ Mcc | 8.5 | 8.56 |
| 2 | 1 | .6 ∗ Mcc | 8.5 | 8.47 |
| 3 | 1 | Mcc | 8.5 | 8.48 |
| 4 | 1 | .6 ∗ Mcc | 8.5 | 8.51 |
| 5 | 2 | .4 ∗ Mcc | 8.5 | 8.49 |
| 6 | 1 | .6 ∗ Mcc | 8.5 | 8.52 |
| 7 | 2 | .4 ∗ Mcc | 8.5 | 8.44 |
| 8 | 1 | .5 ∗ Mcc | 4.25 | 4.25 |
| 9 | 1 | .6 ∗ Mcc | 8.5 | 8.52 |
| 10 | 1 | .6 ∗ Mcc | 8.5 | 8.49 |
| 11 | 2 | .4 ∗ Mcc | 8.5 | 8.45 |
| 12 | 2 | .2Mcc | 4.25 | 4.27 |

Mcc = Moment capacity of the connection.

**Table 4**
Material properties of the specimens

| Material | Yield stress (N/mm$^2$) | Ultimate stress (N/mm$^2$) | Modulus of elasticity (N/mm$^2$) |
|---|---|---|---|
| Beam & Column & Angle | 235 | 420 | 2.06 × 105 |
| Bolts 8.8 | 740 | 866 | 2.06 × 105 |
| Bolts 10.9 | 962 | 1115 | 2.06 × 105 |

portion of its rotational capacity so that a final moment of 850 N m was applied to all the specimens except 8 and 12, which were selected to study the effects of loading method choice. For these specimens, only half of the moment was applied. The loading method and the rotational capacity of the connections are shown in Table 3.

For all the specimens, ambient-temperature material properties were measured using standard tensile coupon tests and cross-sectional dimensions were recorded prior to testing in the furnace. The results are summarized in Table 4.

### 2.3. Temperature distribution

The column and beam in the furnace were wrapped with a 2.5 cm thick ceramic fiber blanket. Only the connection zone, including the angles, the flange and web of the columns in the vicinity of the angles were exposed to the fire. Six armored thermocouples were used to measure environmental temperatures in the furnace. Fig. 2 shows variation against time for average temperature measured during the heating and cooling phase in the furnace and the ISO834 and ASTME119 standard fire temperature curves. It can be seen that the temperature of the furnace followed the standard curves closely.

For Type II specimens (SWW), nine thermocouples were installed on the joints to measure temperature changes during the test. For Type I specimens (SOW), seven thermocouples were used. Fig. 5 shows the position of the thermocouples in both connection types.

The temperature distribution in depth and height of specimen S12 is shown in Fig. 6. Some of thermocouples were damaged during the tests. It can be seen that there were negligible differences in temperature distribution around the joint. The use of non-uniform temperature distributions along the joints makes the analysis more complicated. It was assumed in analysis that the joints were heated uniformly, using average temperatures taken from the thermocouples. These average temperatures are shown in Fig. 7.

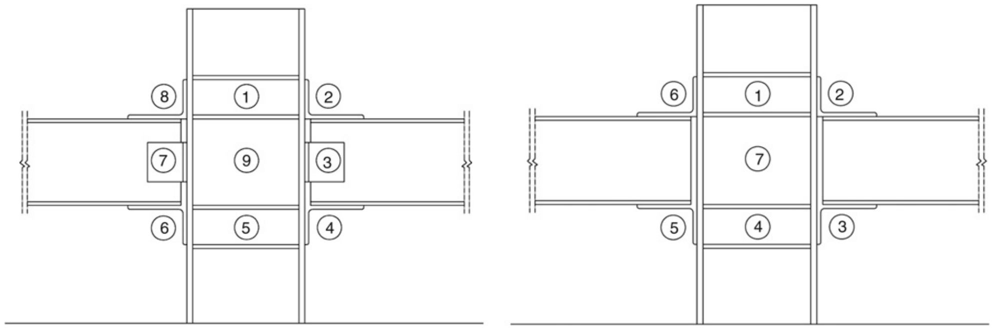



**Fig. 5.** Thermocouple arrangements.

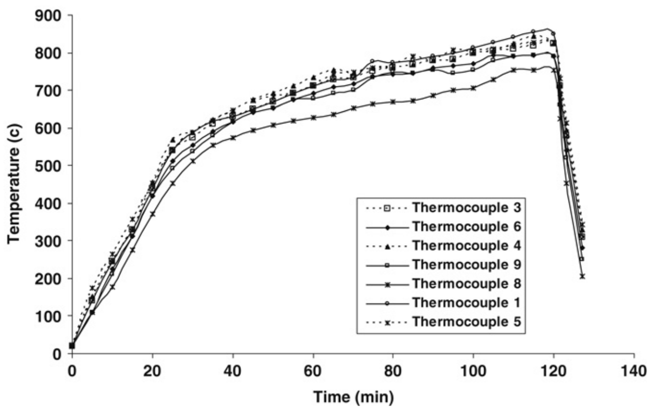



**Fig. 6.** Temperature distribution in specimen 12.

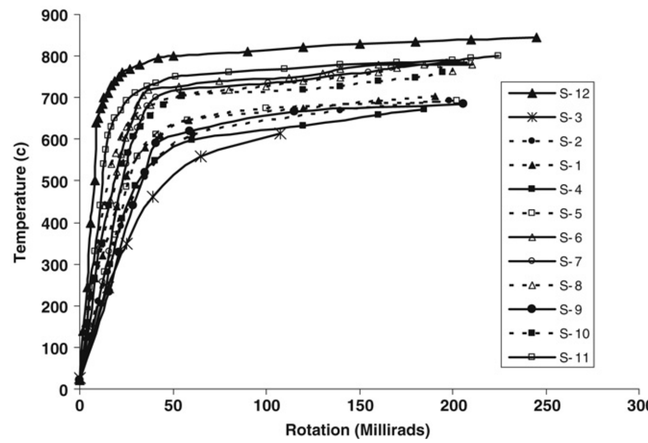



**Fig. 8.** Temperature–rotation response.

## 2.4. Measurement of rotation

Clinometers and LVDT transducers were used to measure the rotation of connections on either side of the beam–column. (Designated as East and West connections). The LVDT transducers were basically used to measure the vertical deflection at different positions along the beam. However, they can also be used to calculate the rotation of the connection. The rotation, $\phi$, based on LVDT transducer readings, can be calculated from the following equation [19]:

$$\phi = \tan^{-1}(\boldsymbol{u}/\boldsymbol{L}) \tag{1}$$

where $u$ is the deflection of the point along the beam and $L$ is the distance from the connection centerline to the point where deflection is measured. There is good agreement between rotations recorded by clinometers and those calculated from the displacement transducer for both the East and West connections. Rotations calculated based on the displacement transducers were used in the tests in which clinometers had been burned during the tests.

## 3. Temperature–rotation response of connections

It is possible to derive temperature–rotation curves of connection at elevated temperatures using the measured data. The temperature–rotation response of connections is shown in Fig. 8.

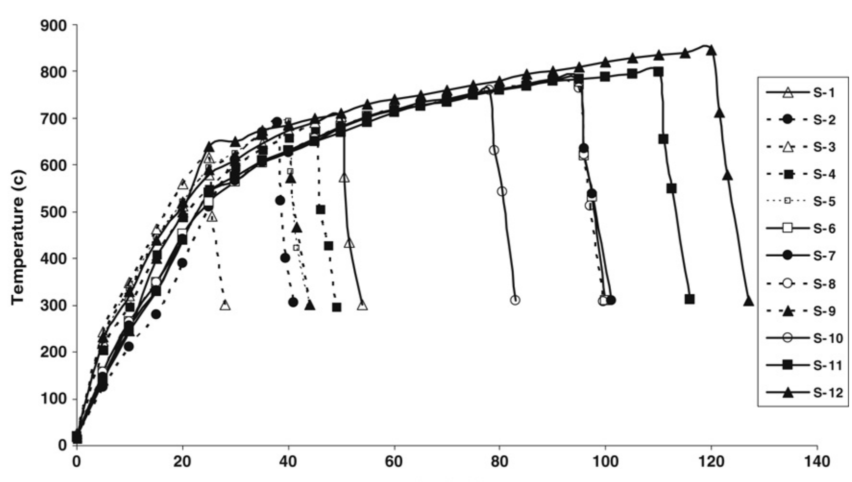



**Fig. 7.** Average temperatures in the specimens.

**Table 5**
The relations between tests

| Specimen number | Group number | Specimen number | Group number | The relations between tests |
|---|---|---|---|---|
| 1 | 2 | 2 | 1 | Effect of Web Angle |
| 5 | 2 | 9 | 1 | Effect of Web Angle |
| 1 | 2 | 5 | 2 | The effect of beam–column flange clearance (gap) |
| 2 | 1 | 9 | 1 | The effect of beam–column flange clearance (gap) |
| 4 | 1 | 3 | 1 | Effect of angle thickness |
| 3 | 1 | 8 | 1 | Effect of the moment value on connection behavior |
| 11 | 2 | 12 | 2 | Effect of the moment value on connection behavior |
| 5 | 2 | 11 | 2 | The effect of bolts |
| 4 | 1 | 6 | 1 | The effect of bolts |

It can be seen that the temperature–rotation curves of the connections can be classified into three regions. Initially there is an approximately linear response with increasing temperatures until one or more components of the connection yield. There is a curved knee that identifies the yielding of the connection. Finally, as the connection failure becomes imminent, the rotation rate increases rapidly causing an almost flat plateau in the connection response.

## 4. Results and discussion

Twelve tests were conducted on two connection configurations and the results were derived as a family of rotation–temperature curves for these connections. In addition to determining the behavior of bolted connections at elevated temperatures, the results can generate criteria for powerful numerical models to study the behavior of these connections. As mentioned before, the configuration of the tests in this study was selected in a way such that it became possible to investigate the effect of certain parameters on the behavior of structural connections at elevated temperatures. To clarify the relations between these tests, the results are summarized in Table 5 for each parameter. It should be noted that the results of this study were obtained using limited tests, and they should be confirmed by further experimentation.

### 4.1. Effect of web angle

The only difference between Groups 1 and 2 was the presence of web angle. The differences between the behaviors of these connections have been studied at ambient temperatures [20, 21]. As seen in Table 5, the effect of the presence of web angle can be studied by comparing the results of four tests at elevated temperatures. The temperature–rotation curves of these connection groups (with and without web angle) are compared in Fig. 9.

As can be concluded from the results, at elevated temperatures, the differences between the groups are negligible and the presence of the web angle does not significantly increase the capacity of the connection to withstand high temperatures. This may occur because the premature tension failure of bolts in these connections does not allow the capacity of the angles to be fully used, particularly for web angles.

### 4.2. Study the effect of beam–column flange clearance (gap)

A clearance of 1–2 cm typically exists between the beginning of the beam and the column flange in the angle connections of steel structures. This gap is generally not considered in the design of connections. In this study, the effect of this gap on the rotational behavior of these connections at elevated temperatures was also

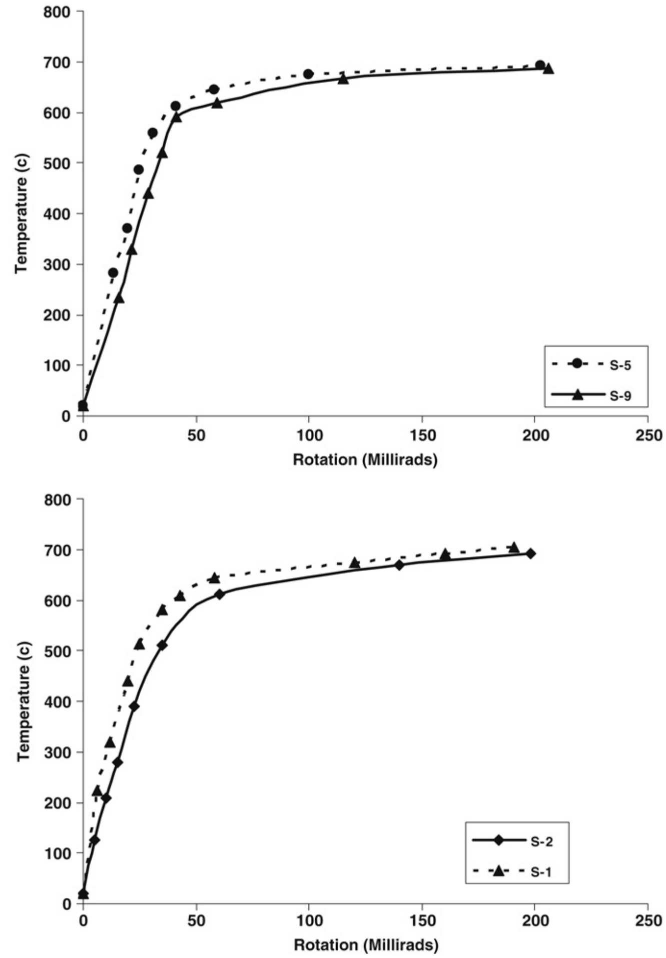



**Fig. 9.** Rotation–temperature curves for connection specimens with and without web angle.

examined. The temperature–rotation curves for the connections with gap compared to those of connections without gap in Fig. 10.

It is evident here that although this gap increases the connection rotation a little in the elastic range, increasing the temperature moves the connection into the plastic range and the increase in connection rotation has no effect on connection behavior. It can be concluded that when the temperature is increased and the connection rotates, the bottom flange of the beam sticks to the column flange and from then on no difference exists between the two specimens.

### 4.3. Effect of angle thickness

The effect of top and seat angle thickness was also studied. The strength and the stiffness of an angle connection directly depend on the strength and stiffness of the angles. The strength and the stiffness of the angles are consequently dependent on the modulus of elasticity and the steel strength of the angles. Increasing the thickness of the angles will increase their stiffness and thus the stiffness of the connection will increase. The modulus of elasticity and the steel strength deteriorate with temperature increases according to Table 6. The full deterioration of stiffness and strength in a connection with higher strength occurs at higher temperatures and so it can tolerate higher temperatures compared to a lower-thickness angle. Specimen S-3, which has lower thickness compared to specimen S-4, showed higher rotations at a constant temperature, but its stiffness and strength were fully deteriorated at a much lower temperature than for specimen S-4.

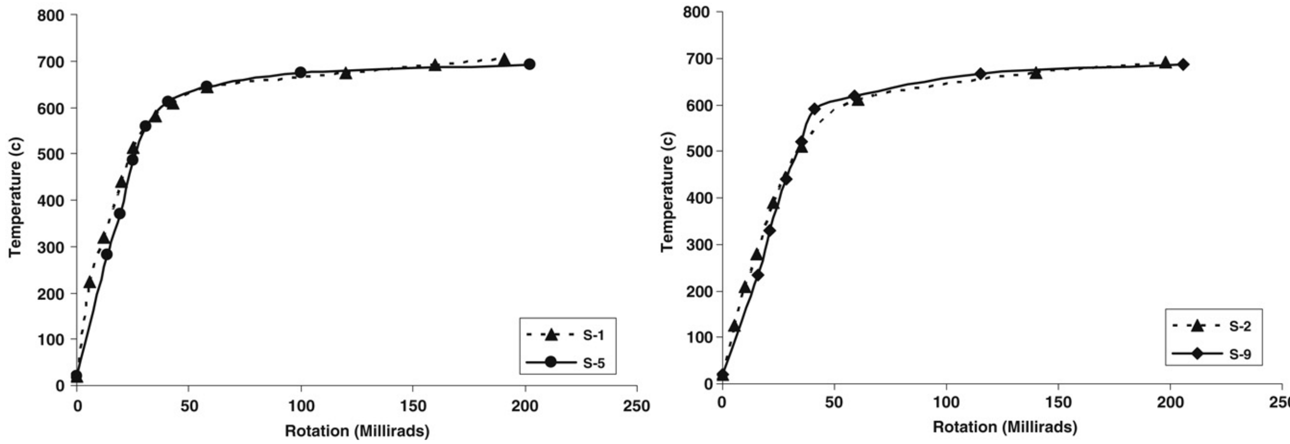



**Fig. 10.** Temperature–rotation curves for the connections with gap and without gap.

**Table 6**
Reduction factors for stress–strain curves of steel at elevated temperatures

| Steel temperature, $\theta_s$ (°C) | Reduction factors for yield stress $f_y$, and Young's modulus $E_s$, at steel temperature $\theta_s$ | |
|---|---|---|
| | $k_{y,\theta} = f_{y,\theta}/f_y$ | $k_{E,\theta} = E_{S,\theta}/E_S$ |
| 20 | 1 | 1 |
| 100 | 1 | 1 |
| 200 | 1 | 0.9 |
| 300 | 1 | 0.8 |
| 400 | 1 | 0.7 |
| 500 | 0.78 | 0.6 |
| 600 | 0.47 | 0.31 |
| 700 | 0.23 | 0.13 |
| 800 | 0.11 | 0.09 |
| 900 | 0.06 | 0.0675 |
| 1000 | 0.04 | 0.045 |
| 1100 | 0.02 | 0.0225 |
| 1200 | 0 | 0 |

The rotation–temperature behavior of specimens S-3 and S-4 are compared in Fig. 11.

### 4.4. Effect of the moment value on connection behavior

The effect of the moment value on connection behavior was also investigated in this study. As seen in Table 2, specimens S-3 and S-8 from the first group and specimens S-11 and S-12 from the second group are exactly the same. According to Table 3, the moment applied to the S-3 specimen was twice the moment applied to specimen S-8, and similarly, the moment applied to the S-11 specimen was twice the moment applied to specimen S-12. The rotation–temperature curves of these two sets are compared in Fig. 12.

As can be observed from the results for these specimens, the connections tolerate higher temperatures with a decrease in moment. Since a load combination consists of the jack load and the temperature load applied to the connection, decreasing the moment will help the connection to tolerate higher temperatures. Although in both tests the applied moment was divided by two, the effect of this parameter was more severe in the S-8 and S-3 specimens compared to S-12 and S-11, as can be seen from Fig. 12. Studies have shown that the stiffness of connections is fully deteriorated at temperatures greater than 900 °C [22] and that common steel connections cannot tolerate temperatures greater than 900 °C. Considering this concept, specimen S-11 tolerated 799 °C before failure, but the S-12 specimen, to which only half the moment was applied, tolerated 850 °C, approximately equal to the maximum tolerable temperature of these connections.

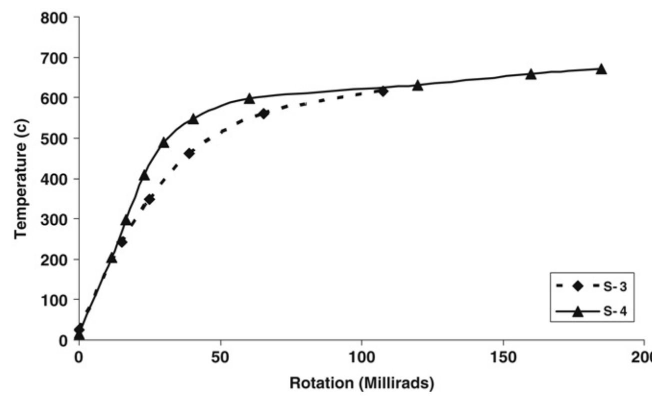



**Fig. 11.** Rotation–temperature curve comparison for the specimens S-3 and S-4.

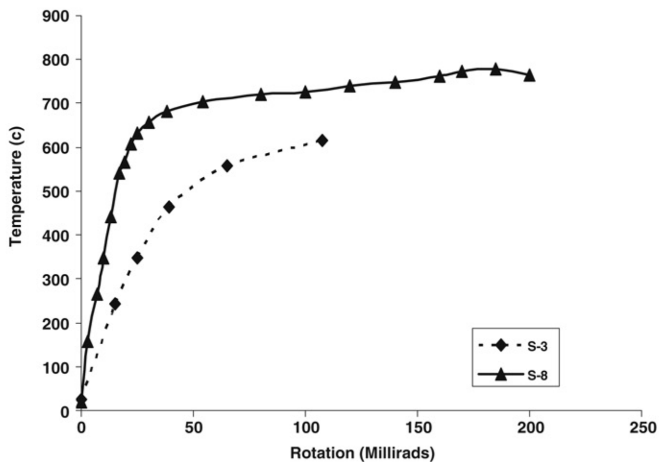
### 4.5. Study of the effect of bolts

One of the most important elements that affect the behavior of connections in fire condition is their bolts. Observations of real steel structures exposed to fire showed that bolted connections show more desirable behavior than welded connections. The main cause of this better behavior is the effect of the bolts in the connections. Studies have shown that the tension and shear strength of bolts dramatically deteriorates at temperatures between 300 and 700 °C. However, the heating rate has no significant effect on bolt capacity. In this study, to investigate the effect of bolt grade on connection behavior, the grade of bolts and nuts were varied in both of the connection groups. As seen in Table 5, specimens S-5 and S-11 from the second connection group and specimens S-6 and S-4 from the first group were selected to study the effect of bolt and nut grade on the connection behavior. As seen in Fig. 13, using 10.9 bolts instead of 8.8 bolts will significantly increase the rotation capacity and the temperature strength of the connection. The reason for this is the better behavior of 10.9 bolts and their higher temperature strength. This confirms the significant effect of bolts on the behavior of connections.

One of the studies carried out to investigate the behavior of bolts at elevated temperatures is the study conducted by B.R. Kerby. In the study, tension tests and double shear tests were conducted on bolts and two major failure modes, tension and shear failure, were studied.

1. Tension tests: The results of the tension tests showed that there are two tension failure modes at elevated temperatures.

The first mode is ductile necking in the threads. The second one is failure due to thread stripping. It should be noted that this



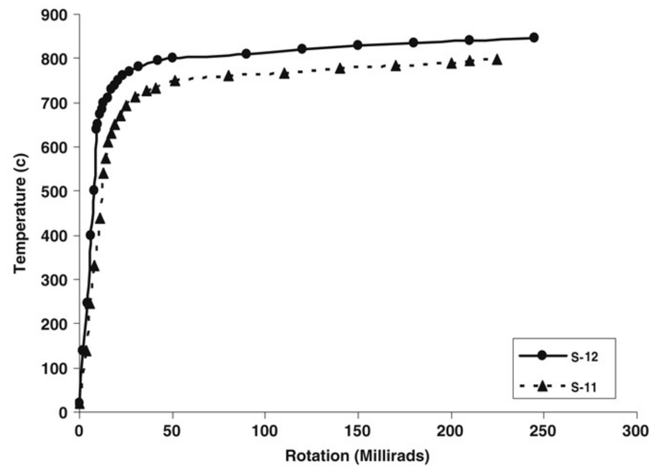
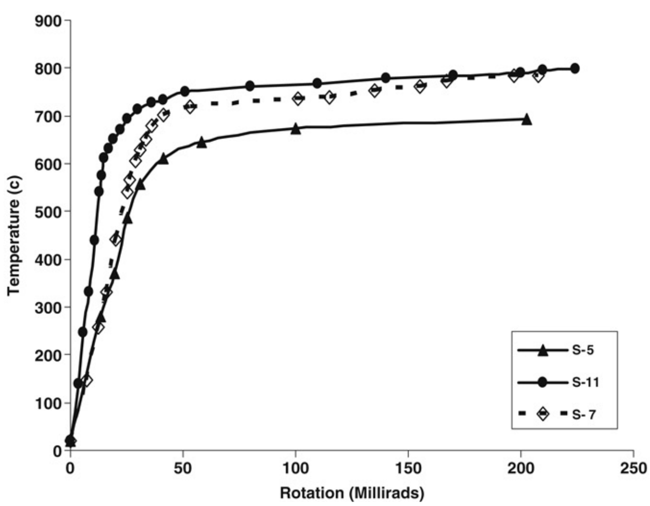




**Fig. 12.** Effect of moment decrease on the rotation–temperature curve of two series of specimens in groups 1 and 2.



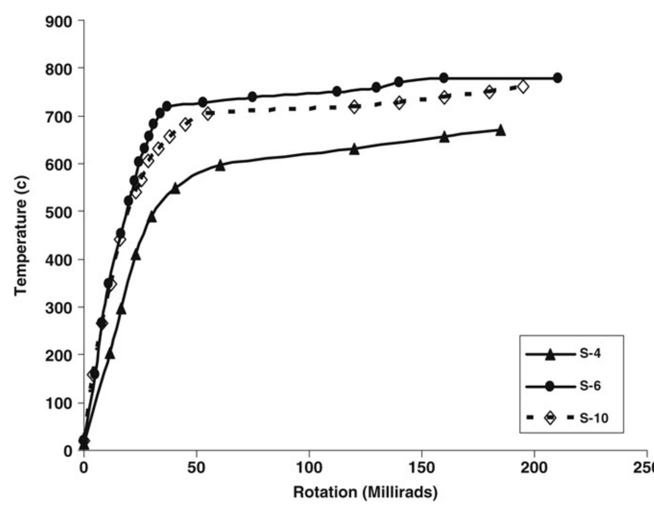




**Fig. 13.** Effect of bolt altering on temperature–rotation curve of two series of connections in connection group 1 and 2.

mode is more frequent than the first one. This was controlled by the interaction of the threads between the nut and bolt rather than the fault of any individual component.

2. Double shear tests: In the shear tests, shear is applied on the bolts in two areas. For the test configurations in which both shear planes acted through the shank, either one or both planes fractured. However, in tests involving shear across the thread, despite this shear path initially having a lower capacity, failure also often occurred in the shank. This was due to the tensile stresses generated by the prying action causing the bolt thread to extend.

The results of [9] were obtained from the tests in which separated bolts were exposed to forces that were similar to the forces existing in a real bolt at elevated temperatures. In the current study, the bolts were studied while in a connection, which is similar to conditions for a real bolt. The results are as follows:

1. All of the 8.8 bolts used with a 8.8 nut failed in tension.

2. The tension failure of 8.8 bolts was often due to thread stripping while the number of cases in which the failure was due to ductile necking in the threads was small. Fig. 14 illustrates the first tension failure in which ductile necking in the treads occurred. Fig. 14a is copied from [9] and Fig. 14b was obtained from the current study. Fig. 15 shows the tension failure mode due to thread stripping.

3. 10.9 bolts with the same grade nuts failed due to shear.

4. In the specimens in which shear failure occurred, the failure was located at the shank of the bolt and no failure occurred at the threads.

It should be noted that since in this study, the bolts were investigated while in a connection, which is similar to conditions for a real bolt, the failure is not a pure shear failure as a result and is occurred due to combinations of tension and shear. But the shear force is prevailing, the failure is considered as a shear failure.

This failure mode is shown in Fig. 16.

5. In specimens S-7 and S-10, to study the feasibility of preventing premature tension failure due to thread stripping, the 8.8 bolt is used with three nuts of the same grade and the results showed that this could prevent premature tension failure. As seen in Fig. 13, specimens S-7 and S-10 demonstrated more desirable behavior and failed at higher temperatures compared to the similar S-4 and S-5 specimens in which the bolts were used with just one nut.

Moreover, in contrast to the failure mode of specimens S-4 and S-5 in which failure was due to thread stripping, the failure mode of the specimens S-10 and S-7 is shear failure at the bolt shank. As can be seen, the behavior of specimens S-7 and S-10 became similar to that of specimens S-6 and S-11 in which 10.9 bolts were used.

The studies carried out by Kerby and the current study obtained similar results for the behavior of bolts at elevated temperatures. These similarities are as follows:

1. Most bolts exposed to tension at elevated temperatures fail due to thread stripping rather than due to ductile necking in the threads.

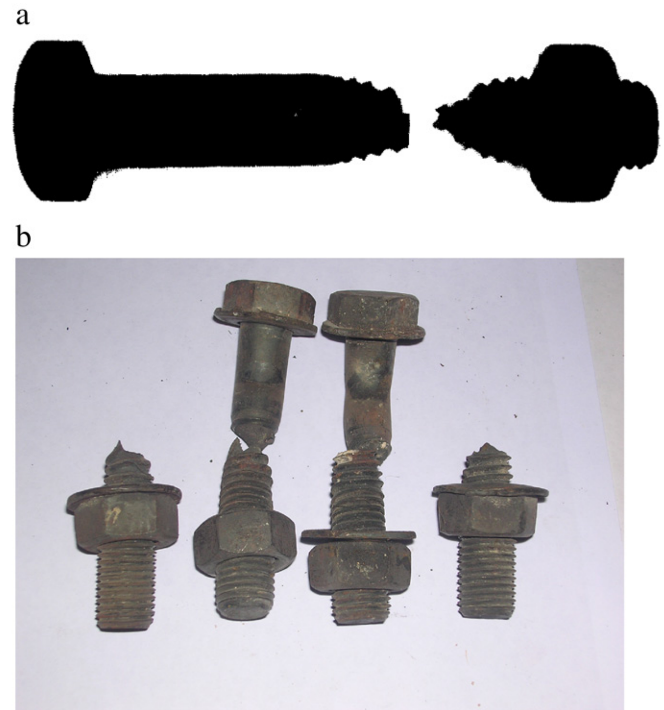



**Fig. 14.** First mode of tension failure of bolts: (a) specimen in Kerby tests, (b) the specimen in current study.

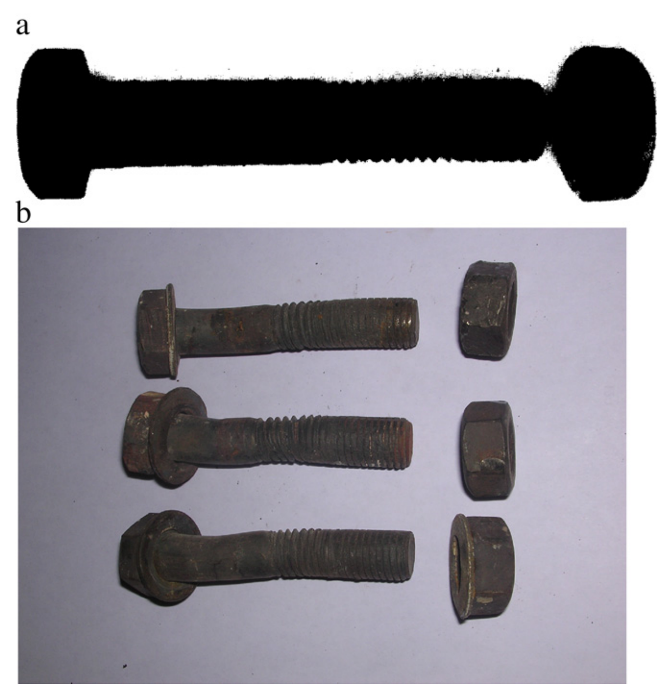



**Fig. 15.** Second mode of tension failure: (a) specimen in Kerby tests, (b) the specimen in the current study.

2. Tension failure due to thread stripping at elevated temperature is common and prevents the complete use of bolt tension capacity.

3. Shear failure is more likely to occur in the shank rather than in the threads.

**Fig. 16.** Shear failure of bolts in the current study.

## 5. Failure modes of connections

According to the test results, the failure mode of connections proceeded as follows: yielding first occurred on the top and seat angles and then extended to the bolts and final failure. Only in one case did the bolts not yield due to angle size. In this case, the failure mode was full yielding of the bottom and top angles and then failure of the top angle while the bolts were still in elastic range. The two failure modes are shown in Figs. 17 and 18.

## 6. Conclusion

To study the behavior of structures at elevated temperatures, twelve experimental tests were carried out on two types of bolted angle beam-to-column connections and a family of rotation–temperature curves was obtained. The results of this study are expected to be useful for other investigations about the behavior of connections in fire, particularly for creating new numerical models since there are few experimental tests in this area. The effects of multiple parameters on the behavior of these two connection groups were studied. From the results obtained, it can be concluded that using temperature-resistant bolts, increasing the thickness of angles and decreasing the applied moment on connections can be used to increase the temperature strength of connections. However, it should be noted that, as the results of this study show, connections made of structural steel fully deteriorate at temperatures greater than 900 °C, and the connection stiffness at this temperature reaches zero.

Furthermore, the results of this study show that bolts are undoubtedly one of the most important components in a bolted connection at elevated temperatures, and improper bolt behavior will prevent the capacity of other components from being completely utilized. Although increasing the stiffness of a connection should increase the temperature strength according to theory, a comparison between the results of the first and second groups showed that an increase in the temperature resistance did not occur. The reason is that premature tension failure of bolts prevented the possibility of using the full stiffness of the connection and web angle effects. Since the effect of this premature failure on the connection capacity is difficult to calculate, it is recommended that components be supplied with at least the dimensional tolerances given in BS3692 and preferably with nuts of a higher strength grade. In situations in which this is not possible, the bolts should be used with two or three nuts. This will prevent premature tension failure of the bolts in many cases according to the results of the studies.

**Fig. 17.** Specimen 5, failure mode I.

**Fig. 18.** Specimen 3, failure mode II.

Considering the presented temperature–rotation curves, all of the curves have elastic behavior until a specific temperature. From when this temperature is reached until the end of the test, the curves follow a plastic deformation curve. The onset of plastic behavior occurs at a temperature between 500 and 650° centigrade. In ASTM E119 code, the top temperature limit for steel beams is 538 °C, assuming that the connections still have half of their yielding strength at this temperature. It can be concluded that (in the limited range of this study) structural connections are not the weak link in braced frames with semi-rigid connections, and that connections fail at the same temperature at which the frame beams are assumed to fail.

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