

Fire performance of bolted connections in laminated veneer lumber (LVL)

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SUMMARY

This paper describes an investigation into the fire performance of bolted tensile connections in laminated veneer lumber (LVL) made from radiata pine. The capacity of the bolted connections depends on the embedding strength of the wood and on the yield moment of the bolts. The purpose of the research was to investigate the relationship between the failure load of LVL timber and the time to failure of the connections when exposed to fire. An experimental investigation was carried out on the axial tensile strength of three types of bolted connection that utilised either wood or steel splice plates. Some specimens were tested at ambient temperature while similar specimens were tested in fire conditions with a constant applied load. In addition, single-bolted connections were tested under constant elevated temperature conditions to determine the embedment strength of the LVL. Connections with no steel plates, or with steel plates slotted between the timber members performed better than those with exposed steel. A simplified design approach based on an extension to the Johansen formulae such that the embedment strength of the LVL depends upon the temperature in the bolt has been proposed for the fire resistance of connections.

1 INTRODUCTION

Fire is unpredictable and dangerous, especially in residential buildings. The effect of fire on timber structural members is very complex because of the large number of variables involved. Once ignition has occurred, then a layer of char forms as the wood burns. A structural wood member will lose load capacity as the wood is converted to charcoal which has no strength. The thickening char layer protects the remaining wood, resulting in a predictable rate of charring below the surface. The rate of development of this charred layer determines how long the member can continue to carry load before the strength of the remaining unburned wood is exceeded. A thin layer of heat-affected wood below the char layer will have reduced strength and stiffness.

In recent years, a number of research papers have been published on the influence of temperature on the mechanical properties of wood [e.g. 1-4]. Research has also been carried out into the performance of joints in timber members when subjected to fire temperatures [e.g. 5- 9]. Particular research into the embedment strength of wood at elevated temperatures has also been carried out by Moraes et al [10, 11]. They carried out embedment tests at temperatures ranging from 20°C to

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240°C. These specimens of timber plus 8 mm diameter dowels were heated for 110 mins in an oven, then placed in a temperature controlled chamber on the test machine and kept at the test temperature for a further 10 mins before testing to a maximum displacement of 5 mm.

This paper describes an experimental investigation carried out to determine the axial tensile strength of fire-exposed LVL members with three types of bolted connection utilising either wood (LVL) or steel splice plates. The arrangements tested were wood-wood-wood (W-W-W), steel-wood-steel (S-W-S), and wood-steel-wood (W-S-W). Some specimens were tested at ambient temperature to determine the ultimate strength, while similar specimens were tested in fire conditions under constant applied load. The purpose of the research was to investigate the relationship between the failure load of LVL timber and the time to failure of the connections when exposed to fire. In addition, single-bolted connections were tested under constant temperature conditions to determine the embedment strength of the LVL [12] over a range of temperatures. The variation in the embedment strength was then used in Johansen's equations (as presented in EC5 [13]) to predict the failure of single bolted connections between LVL members [12] as well as multi-bolted connections [12, 14]. The full test series included bolts, dowels, nails, screws and proprietary connectors but this paper reports only bolted joints, with the tests on others covered elsewhere [14].

2 TESTS CARRIED OUT

Testing was carried out on three arrangements as shown in Figure 1 for both single- and multi-bolted connections. The design of the multi-bolted connection was based on a tensile member in the bottom chord of a floor or roof truss. The timber (LVL) members being joined were 150 x 63 mm. The LVL side plates were 150 x 45 mm. The design properties of the LVL are shown in Table 1. The steel plates (side or central) were 6 mm thick. All bolts were 12 mm diameter and were made of Grade 4.6 steel.

The design load on the joint was taken to be 40% of the ultimate tensile strength of the LVL in cold conditions (i.e. a load of 40% of 221 kN = 88kN) by assuming that other design conditions will be more critical than the tensile strength of the member. With a calculated load factor of 0.33 for fire conditions, this gave an expected fire load of 29kN. Six bolts were used for the W-W-W joint, four bolts for the S-W-S joint, and five bolts for the W-S-W joint. The different connections were fabricated as shown in Figures 2 to 4.

The same size timber and steel members and bolts were used to fabricate single-bolt joints. The bolts were placed on the member centreline with an end distance of 100 mm.

3 COLD TESTING

All the joints were tested at ambient temperatures to ascertain the likely ultimate strength relative to the design load. This testing was carried out in compression, rather than tension. The deflection was measured as the load was applied, with one potentiometer on each side. Typical results are illustrated in Figure 5 and show only a small variation between tests. Failure was caused by longitudinal splitting in the LVL at the bolt positions. A greater end distance could be used to increase the failure load, however this is considered unnecessary as the failure loads in all the tests were already 100 – 150% higher than the design load in the New Zealand timber design code [16].

4 FIRE TESTING

A custom-built testing frame and furnace allowed each specimen to be held under constant load while exposed to simulated fire conditions (Figure 6). Each test specimen was positioned and the air supply was regulated so that the heated specimen was subjected to approximately uniform charring on all sides. Figure 7 shows that the heating from the electric coils in the furnace was not

able to heat the furnace as rapidly as the standard ISO 834 fire curve [17] and the two curves for the furnace tests were typical of the variations that occurred between tests.

Each specimen took between 2 and 5 minutes to begin charring, as evidenced by smoke coming from the furnace. The surfaces of the test specimens were not visible from outside the furnace. This led to a period where the test specimens were charring and building up pyrolyzates within the furnace, but prior to flaming [18]. After a few more minutes, the gases reached their unpiloted ignition temperature and ignited. There were flames on all surfaces of the test specimens from the time of ignition until the conclusion of the test.

When each test specimen had failed, the furnace was switched off and the specimen was quickly removed from the furnace. The flames were then extinguished and the specimen cooled with water to prevent further charring.

4.1 Time to Failure

Each specimen was loaded with a constant load of 29kN before starting the fire test. Time measurement started when the furnace was turned on. The failure time was the time at which the load was not able to be sustained. The results are shown in Table 2. Figures 8 to 10 show typical connections after testing. Not only has the LVL suffered considerable charring, but the bolts have cut elongated slots in the LVL as they heated up during the fire.

4.2 Prediction of Fire Resistance

Because the furnace did not follow the standard ISO 834 fire, the time of failure in the tests was not the same as the fire resistance of the connections. To convert from the time of failure to an estimated fire resistance, the fire severity on the surface of the timber piece and the rate of char were analysed.

During a fire, a layer of char forms over the surface of unburnt timber which then shrinks and burns away after a period of time. The fire resistance of a timber member with no connection is related to the residual cross section after charring. The base of the char layer is at approximately 300°C, with a heated layer below the char front. The part of this layer above 200°C is known as the pyrolysis zone which is undergoing thermal decomposition into gaseous pyrolysis products, accompanied by loss of weight, loss of strength and discolouration [19].

To assess the charring rate of the timber, the char layer was removed after testing and the remaining timber section was measured. The depth of char and rate of char in Table 3 are averages over the duration of the test, which is the observed depth of char divided by the total duration of exposure, including the period at the beginning of the test before the onset of char. This was consistent with the testing procedure of Lane et al. [20] who provided the standard ISO 834 charring data used for comparison by Harris [18] and for the testing reported herein.

The charring rates for the test specimens in the furnace range from 0.53 mm/min up to 0.70 mm/min, with a mean value of 0.65 mm/min. This was slightly lower than the average charring rate observed in the ISO 834 furnace by Lane et al. [20] of 0.72 mm/min. Harris [18] developed a formula to convert the failure time in the furnace to failure time in a standard fire (fire resistance):

$$t_{ISO} = \frac{c_{cust}}{c_{ISO}} t_{cust} \quad [1]$$

where

t_{ISO} = Fire resistance time in ISO furnace (minutes)

t_{cust} = Time to failure in the custom furnace

c_{ISO} = Char rate recorded in the ISO furnace (0.72 mm/min)

c_{cust} = Char rate recorded in the custom furnace (mm/min)

The actual durations in the furnace were converted to expected durations in the ISO fire, or the fire resistance. The results are shown in Table 3.

The fire resistance of the connections was also determined using the “total radiant heat exposure” concept suggested by Nyman [21]. In this method, the measure of fire severity is the cumulative radiant heat exposure at any time. The radiant heat exposure in the test fire can be compared with that in the ISO 834 fire to give an equivalent fire resistance. For example, in Figure 11 the fire severity curve for the connection is similar in shape to the fire severity curve of the ISO 834 fire curve, and the test connection failed at 22.5 minutes which is equivalent to 20.5 minutes exposure to the standard fire.

The comparison between the fire resistance found using the mean charring rate and the fire severity correlation is also shown in Table 3. There is only a slight difference in fire resistance of the connections found from the charring rate and that found using the fire severity correlation.

König [22] states that for connections with side members of wood, i.e. the W-W-W and W-S-W connections, fire resistance durations of 15 minutes for bolted connections is achievable. This can be seen to be in line in this research where the fire resistance of a W-W-W joint was estimated to be 21.2 minutes and that of a W-S-W joint to be 16.3 minutes.

5 HEATED TESTING

In order to develop a simple method of predicting the load capacity and deformation of connections in timber structures when exposed to known heat flux, a series of tests were carried out at known temperatures in a similar manner to that outlined in previous research [10, 11]. For this testing, a series of single-bolt joints were heated in the furnace for two hours at a constant temperature with no applied load under temperatures ranging from ambient to 250°C, then quickly loaded to failure.

5.1 *Embedment Strength versus Temperature*

Since the S-W-S connections were similar to standard embedment specimens, the results from these tests were used to estimate the embedding strength for the LVL at elevated temperatures. The main differences between the single-bolt joints tested and the testing apparatus as required by ISO 10984-2 [23] are outlined in Table 4.

In the ISO standard, the “embedding strength” is based on either the maximum load or the load carried at 5 mm displacement, depending on which occurs first. As the bolted connection tested contained two bolts (one at each member end as shown in Figure 12) and the maximum load occurred at a large displacement, the embedding strength was calculated by dividing the load at 10 mm displacement by the bolt diameter and the thickness of the member. Figures 13-15 show the load-displacement relationships for S-W-S, W-S-W and W-W-W connections after correcting for initial slip. The joint strengths at different temperatures were then substituted into the relevant Johansen yield formulae as given in EC5 [13] to determine the embedment strengths. Other information required included the bending strength of the bolt at the various temperatures, the timber thickness, and the experimental failure mode. The results for the calculated embedment strength are shown in Figure 16 where it can be seen that the embedment strength decreases as the temperature increases, reaching a minimum at about 120°C and then increasing as the temperature increases further to 200°C. The reason for this increase is not clear and it is suggested that for design purposes the embedment strength should conservatively be taken as the lower value given by the dashed line until the reliability and extent of this increase in embedment strength can be assured by further testing.

5.2 Prediction of failure load versus temperature

The temperatures of the air, steel plates, and at several points on the bolts were measured using thermocouples and are shown in Figure 17 for the S-W-S connection during the fire test. It can be seen that the temperatures of the steel side plates and the bolts are effectively the same.

Figure 18 shows the temperature of the air, the steel plates, and at several points on the bolts for the W-S-W connections during the fire test. It can be seen that the temperature of the bolt head and at the middle of the side members were similar (leaving aside the discrepancy in one thermocouple), while the temperature in the bolt at the steel plate is considerably lower but similar to those in the timber members. In the case of the W-W-W connections, Figure 19 shows that the temperature varies considerably throughout the joint.

Using the embedment strength determined in Section 5.1, the failure loads predicted by each of the European yield formulae [13] at each temperature were calculated for the single-bolt S-W-S, W-S-W and W-W-W connections and are shown in Figures 20-22, together with the experimental failure loads for each relevant failure mode. All the possible failure modes for each connection type are shown in Table 5, along with the relevant Johansen yield equation.

For the S-W-S connections, the experimental failure mode for the bolt temperatures up to 120°C was mode m whereas for temperatures above 150°C the failure mode was mode k. Figure 20 shows the predicted failure loads for modes j/l and k to be conservative for temperatures up to 120°C as they predict lower failure loads than were observed. For temperatures above 150°C, the predictions for failure in modes j/l and m are higher than found experimentally.

For WSW connections, the predictions in Figure 21 show that up to a bolt temperature of 150°C, Failure Mode g is the most probable failure mode. The experimentally observed failure mode agreed with the predicted failure mode. The predicted failure load was also conservative because the failure load for Failure Mode g using Johansen's Equation is always lower than the actual experimental load.

The prediction of the WWW connections using Johansen's Equations in Figure 22 shows that up to a bolt temperature of 60°C, Failure Mode k is the most probable failure mode whereas beyond a bolt temperature of 60°C, the most probable failure mode is Failure Mode h. The experimentally observed failure mode showed that Failure Mode k occurred over the range from 20°C to 140°C. This partially agreed with the predicted failure mode. At higher temperatures, Failure Mode j occurred rather than the predicted Failure Mode h. It can be seen that Johansen's equations over predicted the failure load up to 60°C and under predicted at bolt temperatures higher than 60°C. This shows that the predictions at higher temperature were conservative.

6 PREDICTION OF LOAD CAPACITY VERSUS TIME

Using the experimental embedment strength calculated above, the predicted failure loads in simulated fire tests for the three types of connections tested by Lau [14] (Figures 2-4) are shown in Figures 23-25.

Initially, the contact thickness between the bolt and the timber members was taken as the original thickness less the thickness of the charred surface as indicated in Figure 25a, given by:

$$t_{contact} = t - (n \times D \times \theta) \quad [2]$$

where t = Original timber thickness (mm)

n = Number of charring surfaces (-) ($n = 2$ for SWS, otherwise $n = 1$)

D = Experimental charring rate (mm/min)

θ = Charring duration (min)

The failure loads predicted by the failure formulae were compared with the experimental failure results from fire tests carried out by Chuo [12] on both single-bolt and multi-bolt connections. These comparisons showed some large discrepancies and as a result a modified model was proposed wherein a char rounding effect at the edges of the bolt holes was included. This effect was observed in the test specimens at the end of the fire test and is illustrated in Figure 26b with the bolt to timber contact length being given by:

$$t_{contact} = t - (2n \times D \times \theta) \quad [3]$$

where the quantities have the same meaning as those for Equation 2, and the factor 2 accounts for the char rounding effect at the edges of the bolt hole.

The predicted failure loads using the temperature dependent embedment strength and Equation 3 for the three types of connection as tested by Lau [14] are shown in Figures 23-25 with several different failure modes for each prediction, Figures 23-25 show that the prediction of the failure mode and the failure load using the bi-linear embedment strength curve, together with the experimental charring rate and steel strength reduction factor for temperature, was reasonably accurate. However, the estimation of the failure time was less accurate, particularly for the S-W-S connection. This is because after the LVL had reached its constant embedment strength at 120°C (Figure 16), the load carrying capacity of the connection reduces only slightly as the timber chars. Therefore, the predicted failure time is very sensitive to the predicted load level.

Figures 23-25 also show the total joint displacement that took place during the fire tests. Failure of the joints was due to the rapid increase in displacement that took place.

7 CONCLUSIONS

- Of the three types of connection tested in compression at ambient conditions, the W-W-W and W-S-W connections had similar ultimate strength and the S-W-S connections were found to have lower ultimate strength.
- In the ambient tests, failure was caused by longitudinal splitting in the LVL at the bolt positions. A greater end distance could be used to increase the failure load, however this is unnecessary as the failure loads in all the tests were 100 – 150% higher than predicted by the New Zealand timber code.
- During furnace testing, the connections with mostly wood exposed to the fire (W-W-W and W-S-W) lasted much longer than those with large areas of exposed steel (S-W-S) where there was higher heat transfer into the connection via the steel side plates.
- The failure times in the non-standard furnace testing were converted to standard fire resistance times by two methods which gave similar results. One method was based on comparing measured char depths and the other was based on the total radiant heat exposure concept.
- The embedment strength of LVL can be described by a bi-linear relationship that varies linearly from 0.08 kN/mm² at 20°C to 0.025 kN/mm² at 120°C, and remains constant for temperatures above 120°C.
- This bi-linear embedment strength was used in conjunction with Johansen's yield equations to predict the failure load and the results showed reasonable agreement with the experimental values.
- Due to the high thermal conductivity of steel, there was very little difference in temperature at different locations along the bolts when the connection is exposed to high temperatures. The average timber member temperatures were generally lower than the bolt temperatures due to the

poor heat conduction of wood. The exception is the S-W-S connection where the timber and bolt temperatures are comparable.

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Table 1 Limit state properties for design with Hyspan [15].

| Elastic Moduli | | |
|------------------------------------|-----------------|-----------|
| Modulus of elasticity | E | 13200 MPa |
| Modulus of rigidity | C | 660 MPa |
| Characteristic Strength | | |
| Bending | f _b | 42 MPa |
| Tension parallel to grain | f _t | 27 MPa |
| Compression parallel to grain | f _c | 34 MPa |
| Shear in beams | f _s | 4.5 MPa |
| Compression perpendicular to grain | f _p | 12 MPa |
| Shear at joint details | f _{sj} | 4.8 MPa |

Table 2 Average time to failure for bolted connections in fire testing

| Connections | Time to charring (min) | Time to ignition (min) | Time to failure (min) |
|-------------|------------------------|------------------------|-----------------------|
| W-W-W | 2.9 | 4.5 | 22.5 |
| S-W-S | 3.3 | 4.5 | 11.4 |
| W-S-W | 3.1 | 4.3 | 19.3 |

Table 3 Calculated fire resistance using the charring rate method

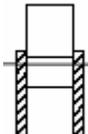
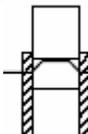
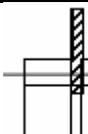
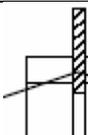
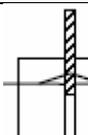
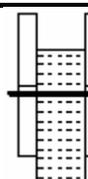
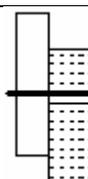
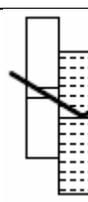
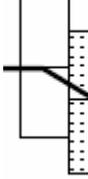
| Connections | Time to Failure (min) | Depth of Char (mm) | Average Char Rate (mm/min) | Calculated Fire Resistance (mins) | Fire Resistance from Fire Severity Correlation (mins) | Difference in Fire Resistance (mins) |
|-------------|-----------------------|--------------------|----------------------------|-----------------------------------|---|--------------------------------------|
| W-W-W | 22.5 | 15 | 0.67 | 21.2 | 20.5 | 0.7 |
| S-W-S | 11.4 | 6.5 | 0.57 | 9.2 | 8.8 | 0.4 |
| W-S-W | 19.3 | 11.5 | 0.60 | 16.3 | 16.5 | - 0.2 |

Mean char rate **0.65**
(mm/min)

Table 4 Comparison of single-bolt connection and standard embedment test

| | Single-bolted SWS connection | Standard Embedment Test |
|----|---|---|
| 1. | Steel members tightly bolted to timber member. | No contact between steel members and test specimen. |
| 2. | Two fasteners were used in each test (i.e. one at each member end). | Only one fastener used in test. |

Table 5 Johansen's yield equations [13] for resistance per shear plane per fastener

| Connection | Failure mode | Equation |
|------------|--|---|
| | k  | $F_{v,Rk} = 1.25 \left\{ 1.15 \sqrt{2M_{y,Rk} f_{h,2,k} d} \right\}$ |
| S-W-S | j or l  | $F_{v,Rk} = 0.5 f_{h,2,k} t_2 d$ |
| | m  | $F_{v,Rk} = 1.25 \left\{ 2.3 \sqrt{M_{y,Rk} f_{h,2,k} d} \right\}$ |
| | f  | $F_{v,Rk} = f_{h,1,k} t_1 d$ |
| W-S-W | g  | $F_{v,Rk} = 1.25 \left\{ f_{h,1,k} t_1 d \left[\sqrt{2 + \frac{4M_{y,Rk}}{f_{h,1,k} d t_1^2}} - 1 \right] \right\}$ |
| | h  | $F_{v,Rk} = 1.25 \left\{ 2.3 \sqrt{M_{y,Rk} f_{h,1,k} d} \right\}$ |
| | g  | $F_{v,Rk} = f_{h,1,k} t_1 d$ |
| | h  | $F_{v,Rk} = 0.5 f_{h,2,k} t_2 d$ |
| W-W-W | j  | $F_{v,Rk} = 1.25 \left\{ 1.05 \frac{f_{h,1,k} t_1 d}{2 + \beta} \left[\sqrt{2\beta(1 + \beta) + \frac{4\beta(2 + \beta)M_{y,Rk}}{f_{h,1,k} d t_1^2}} - \beta \right] \right\}$ |
| | k  | $F_{v,Rk} = 1.25 \left\{ 1.15 \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2M_{y,Rk} f_{h,1,k} d} \right\}$ |

Note: See Table 5A for description of variables used above.

Table 5A List of variables used in Johansen's Equations

| Variables | Descriptions | Units |
|---------------------------------------|---|--------------------|
| t | Timber thickness | mm |
| f_h | Embedding strength | kN/mm ² |
| $\beta = \frac{f_{h,2,k}}{f_{h,1,k}}$ | Characteristic embedding strength ratio | [-] |
| d | Fastener diameter | mm |
| M_y | Fastener yield moment design value | kNmm |
| $F_{v,Rk}$ | Resistance per shear plane per fastener | kN |
| Subscript 1 | Side members | [-] |
| Subscript 2 | Centre member | [-] |
| Subscript k | Characteristic value | [-] |

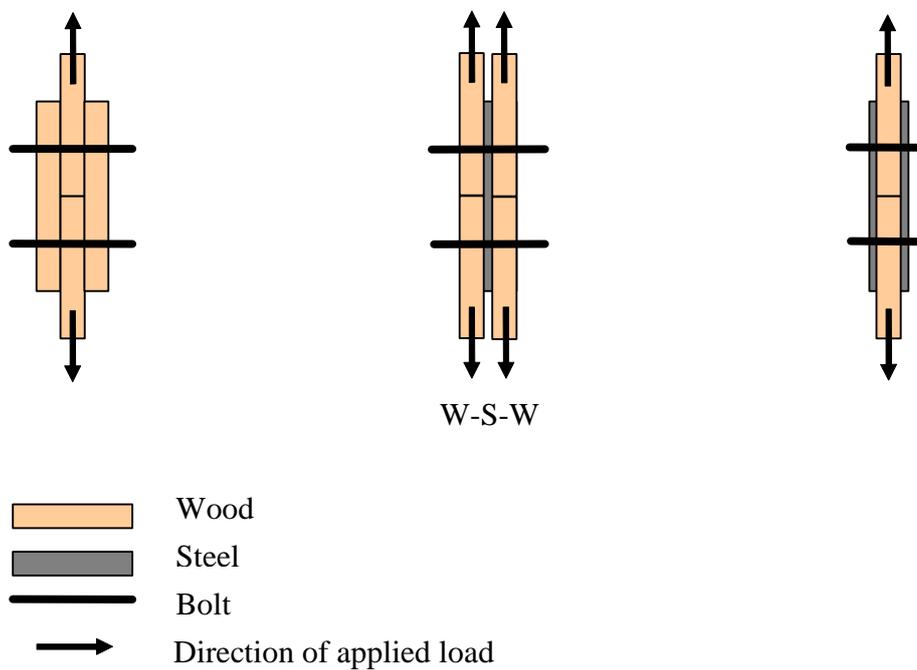


Figure 1 Joint arrangements as tested

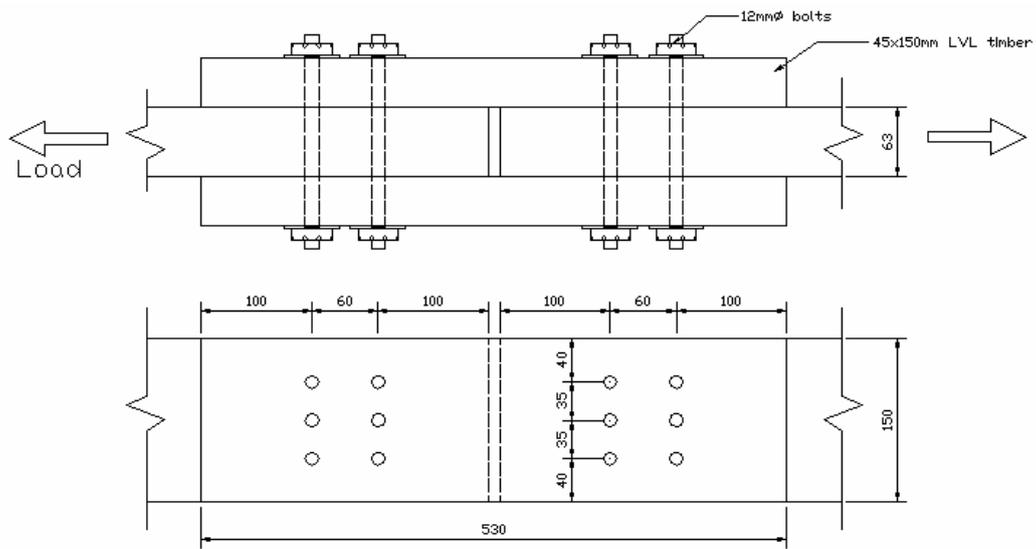


Figure 2 Arrangement of bolted connections for W-W-W tests

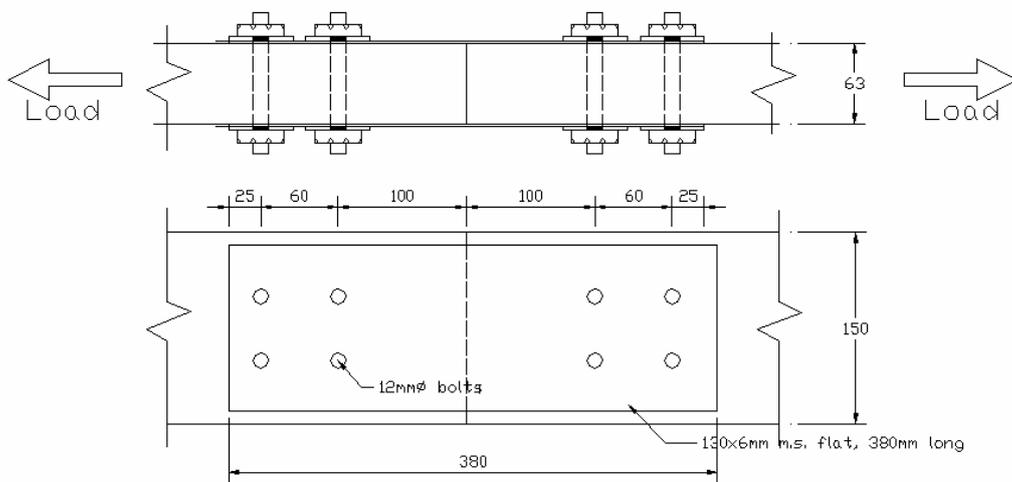


Figure 3 Arrangement of bolted connections for S-W-S tests

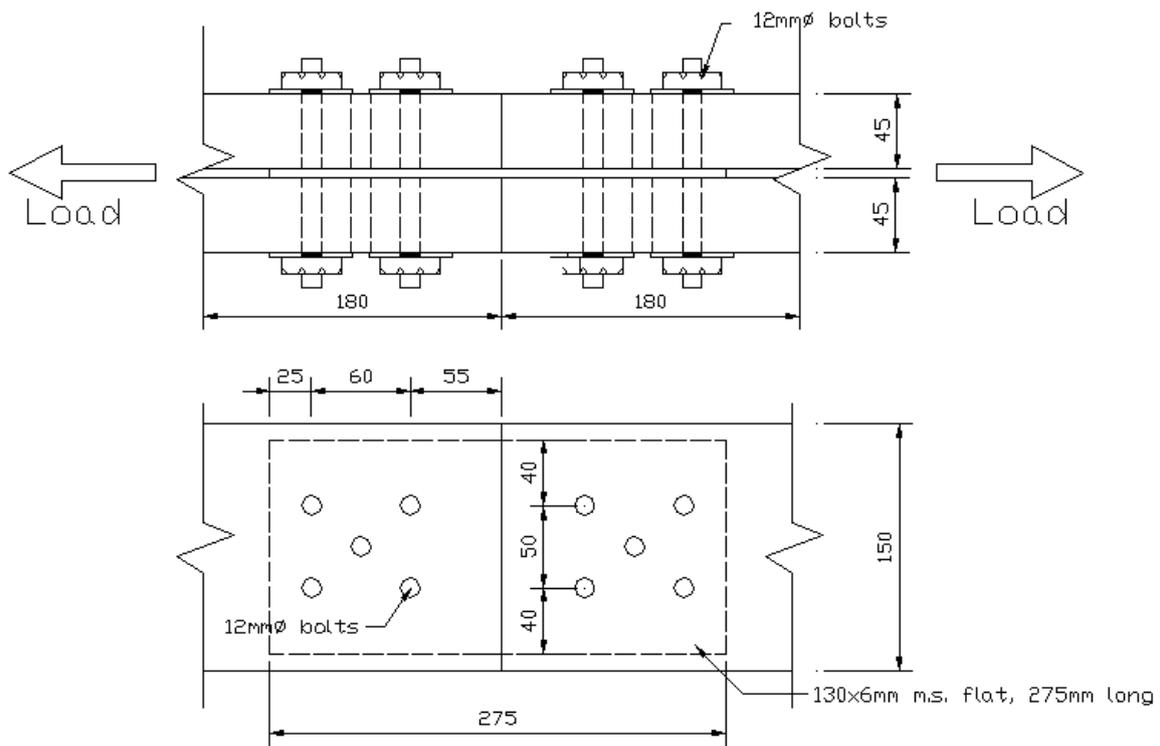


Figure 4 Arrangement of bolted connections for W-S-W tests

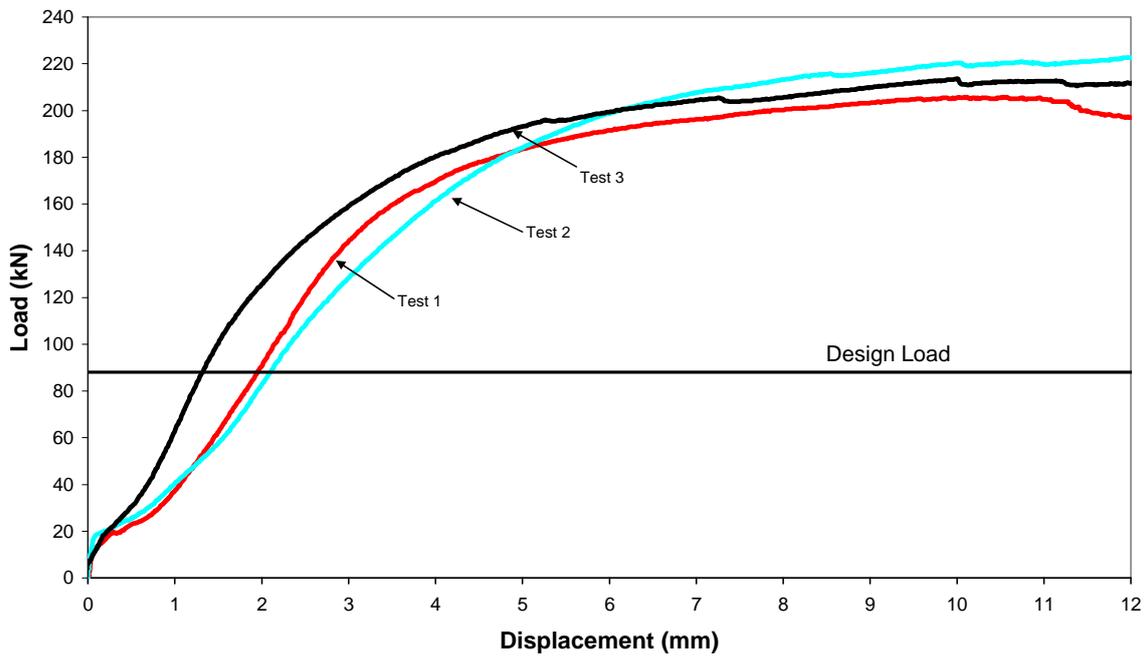


Figure 5 Strength of three W-S-W bolted joints at ambient temperature



Figure 6 The test frame and furnace used for fire testing

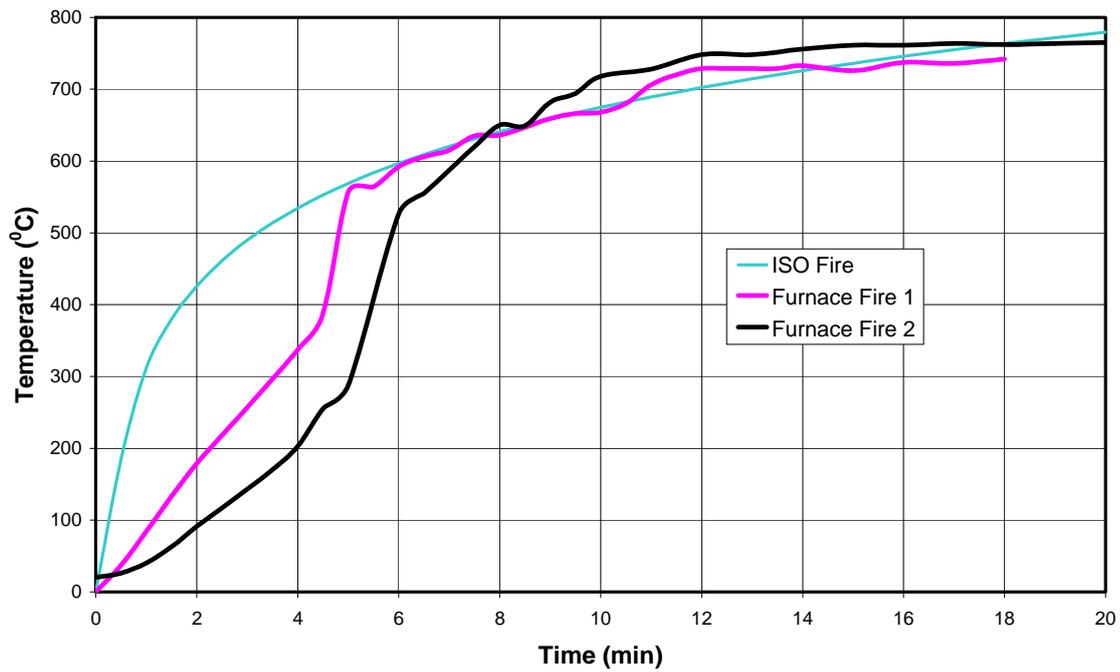


Figure 7 Comparison between ISO 834 fire curve and furnace temperatures for two typical tests



Figure 8 Condition of W-W-W bolted connection after fire tests



Figure 9 Failure mode of S-W-S bolted connection after fire tests



Figure 10 Failure mode of W-S-W bolted connection after fire tests

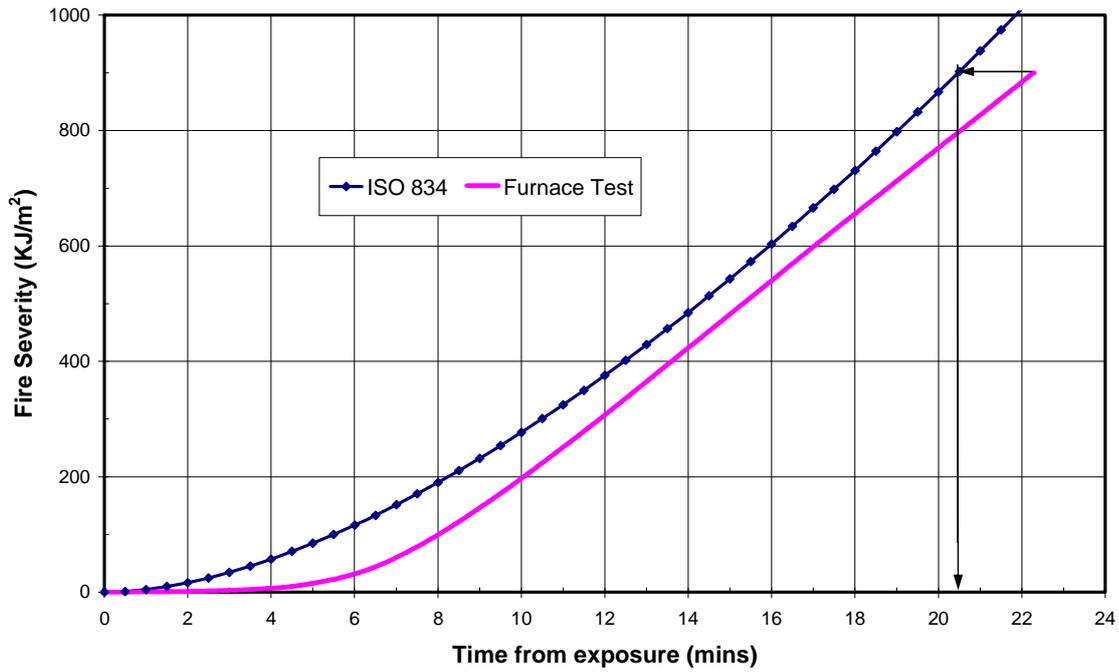


Figure 11 Correlation between time of exposure and fire severity for W-W-W bolted connection. Upper curve shows ISO fire severity and lower curve shows the custom furnace exposure.

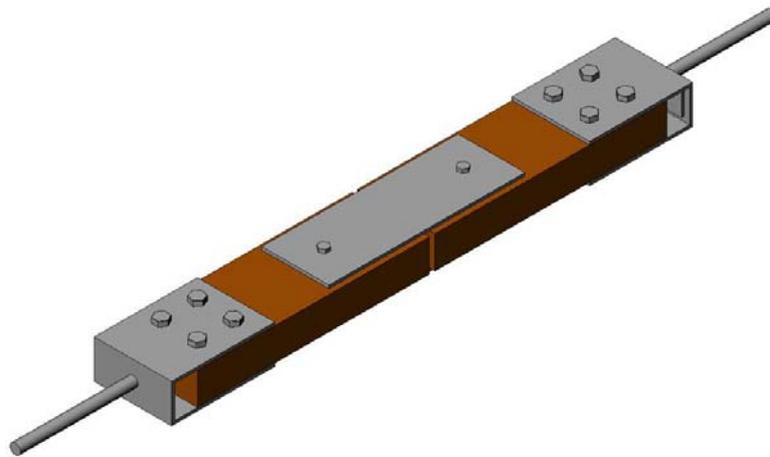


Figure 12 The single-bolt S-W-S connection used for the heated and fire tests

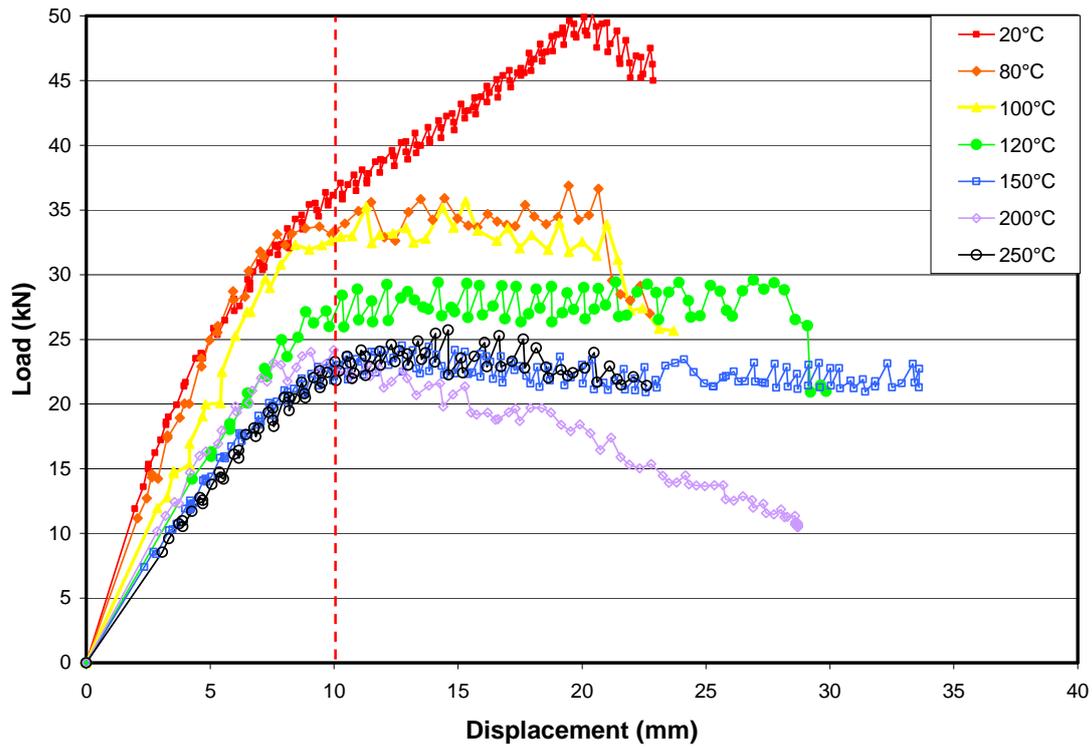


Figure 13 Load-displacement curves for single-bolted joints in a S-W-S connection at various temperatures

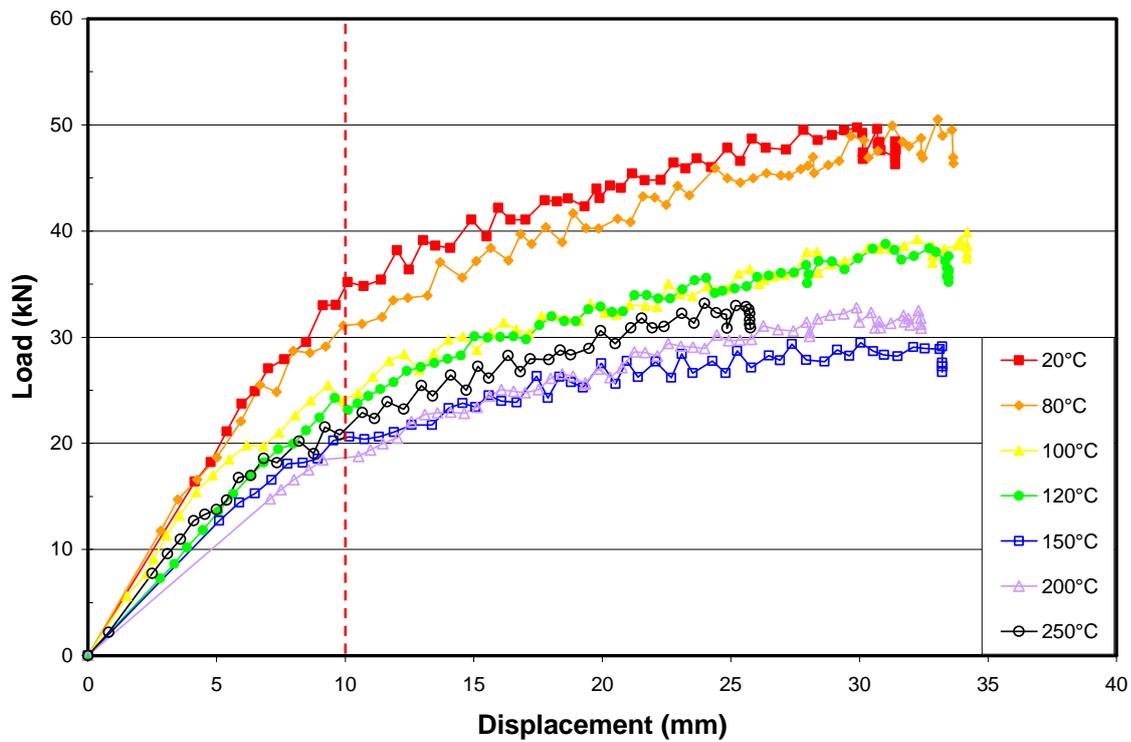


Figure 14 Load-displacement curves for single-bolted joints in a W-S-W connection at various temperatures

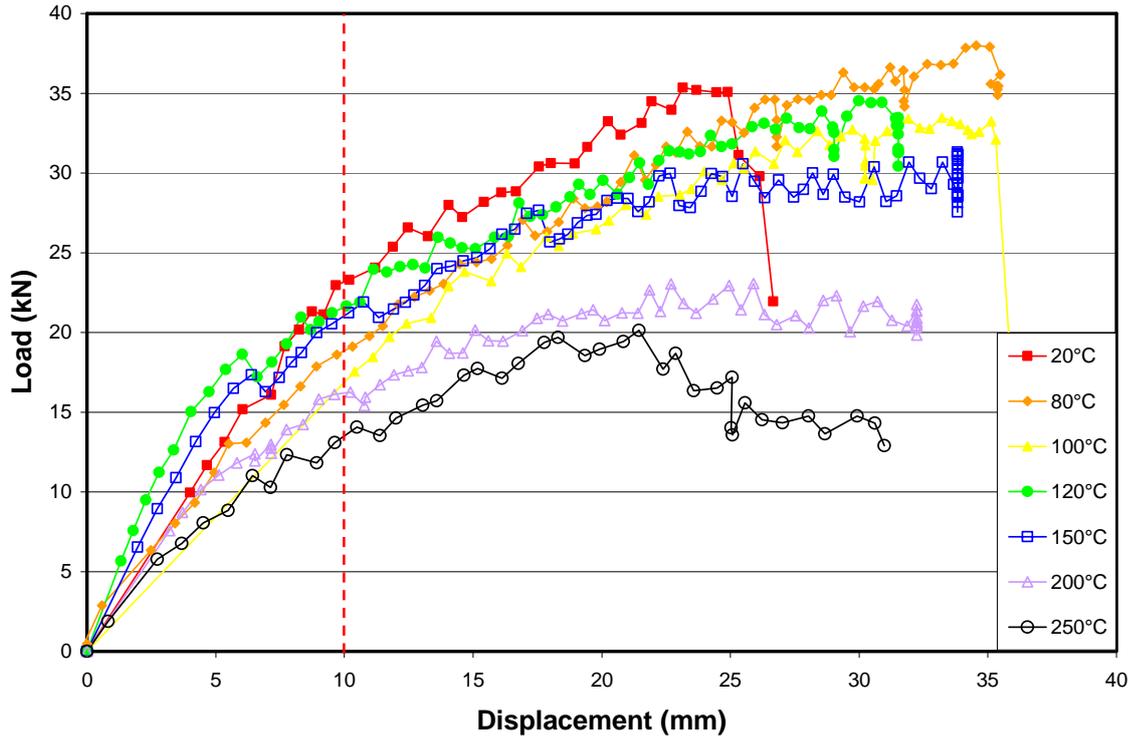


Figure 15 Load-displacement curves for single-bolted joints in a W-W-W connection at various temperatures

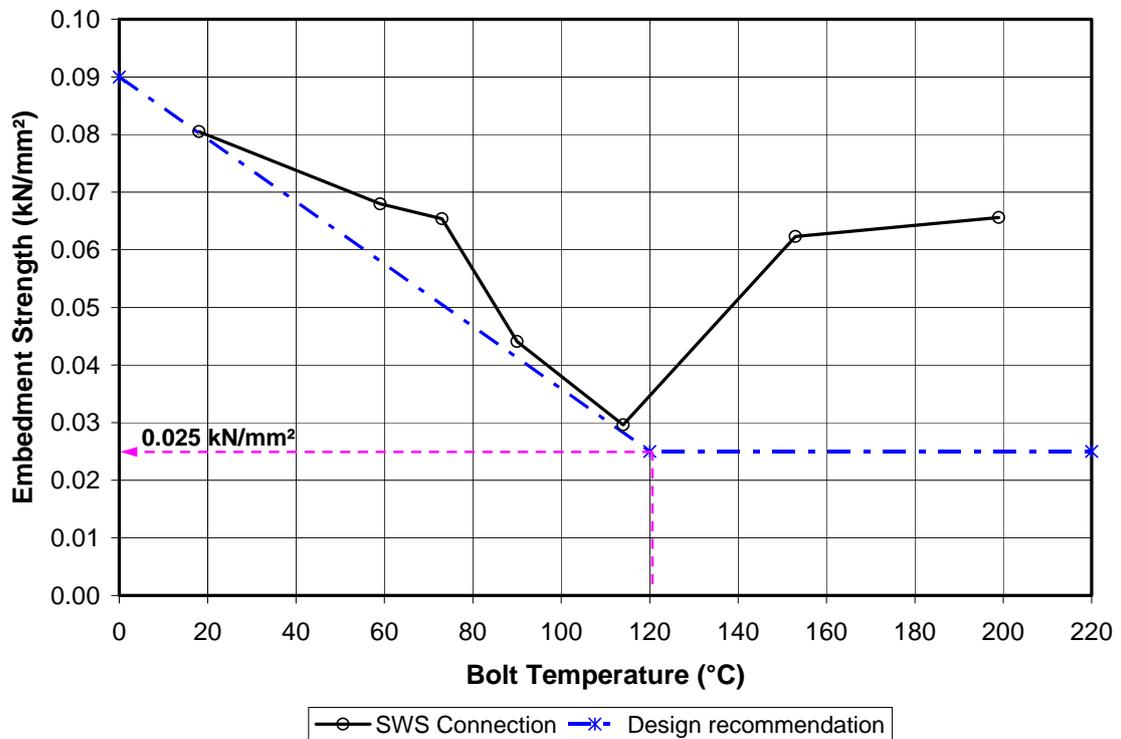


Figure 16 Experimental and modified embedment strength for LVL

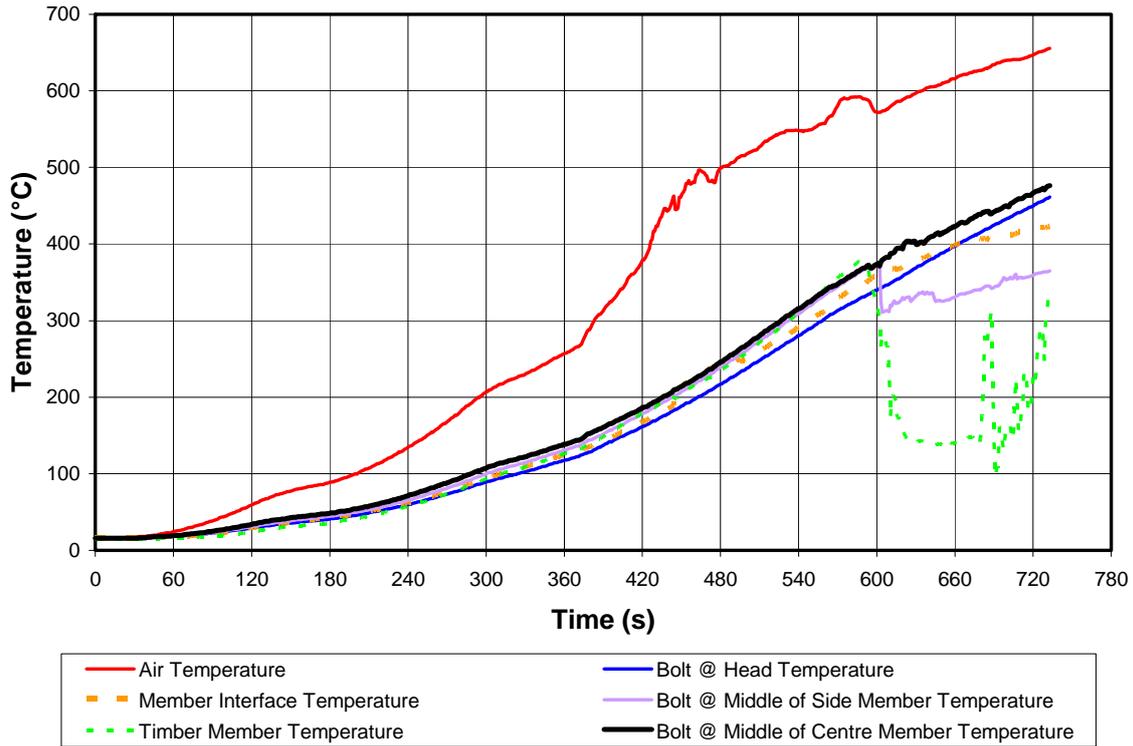


Figure 17 Temperatures measured within the S-W-S connection during the fire test

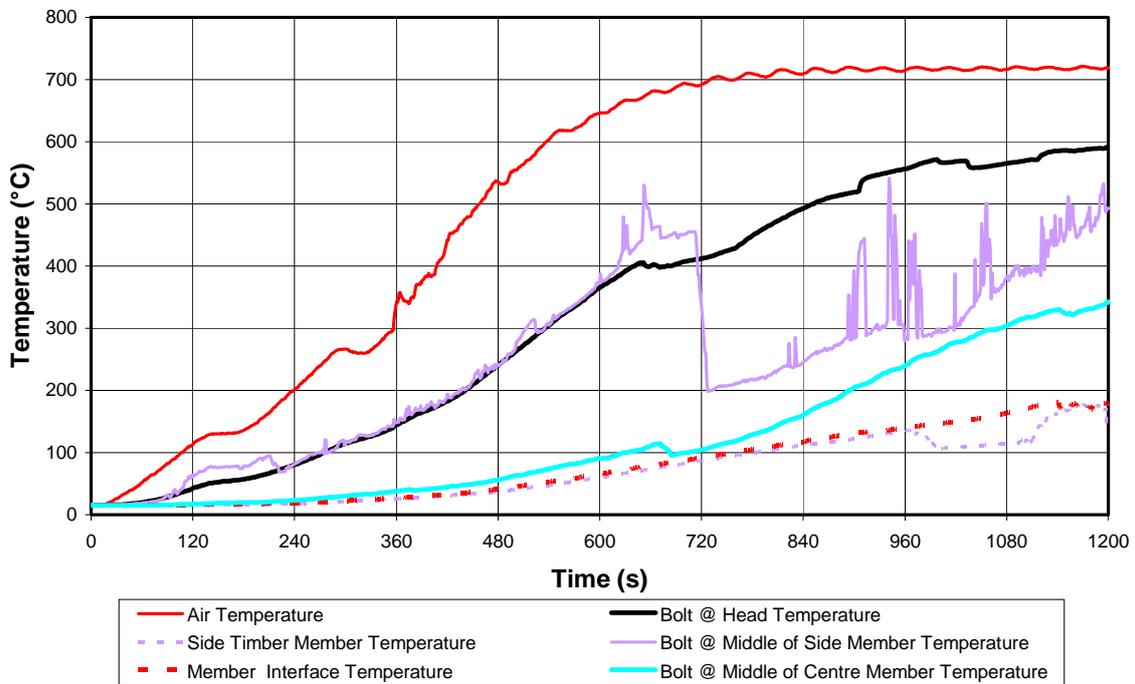


Figure 18 Temperatures measured within the W-S-W connection during the fire test

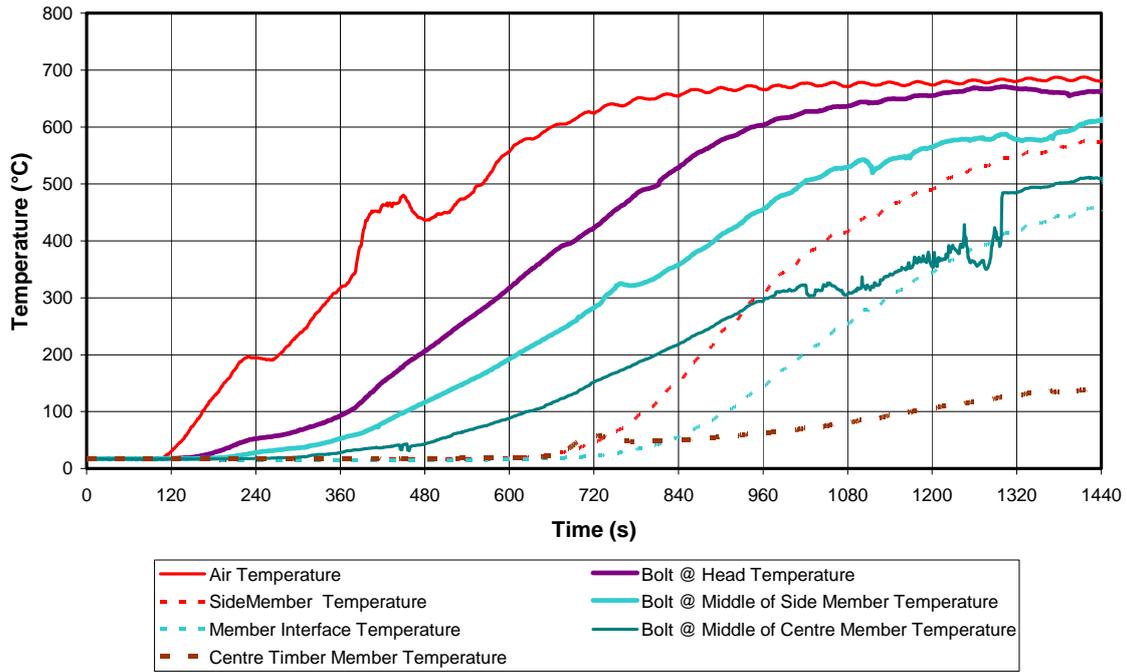


Figure 19 Temperatures measured within the W-W-W connection during the fire test

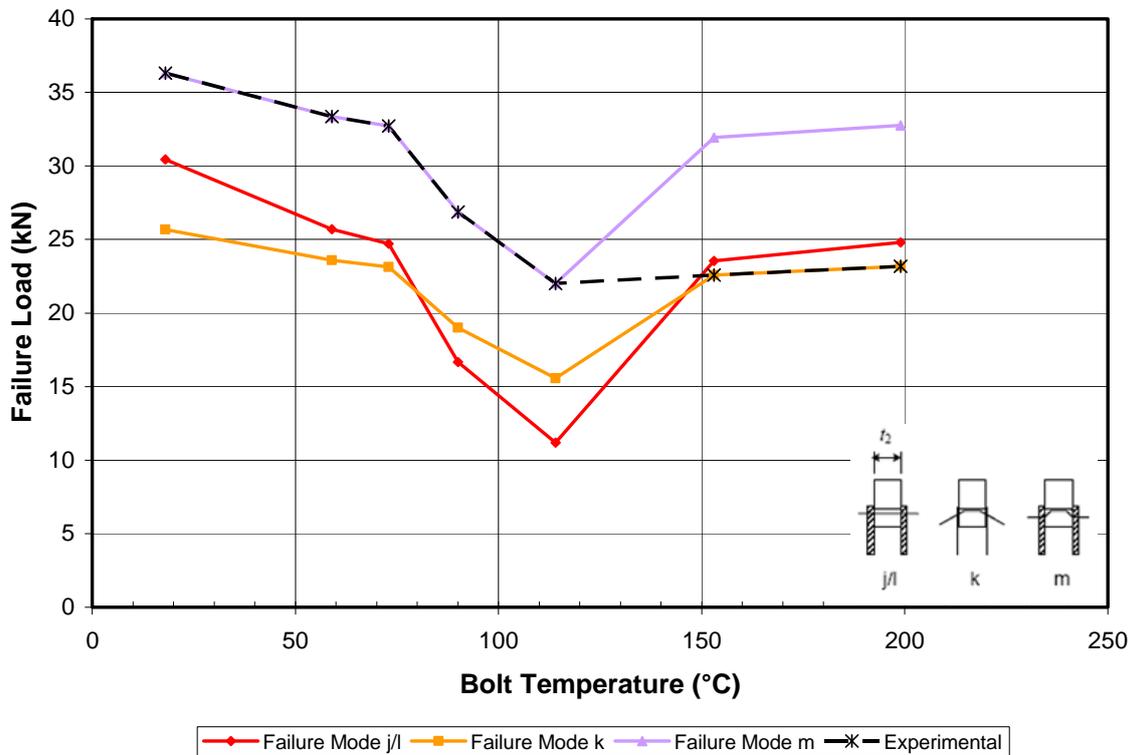


Figure 20 Predicted and experimental failure loads for single-bolt S-W-S connections tested at constant temperature.

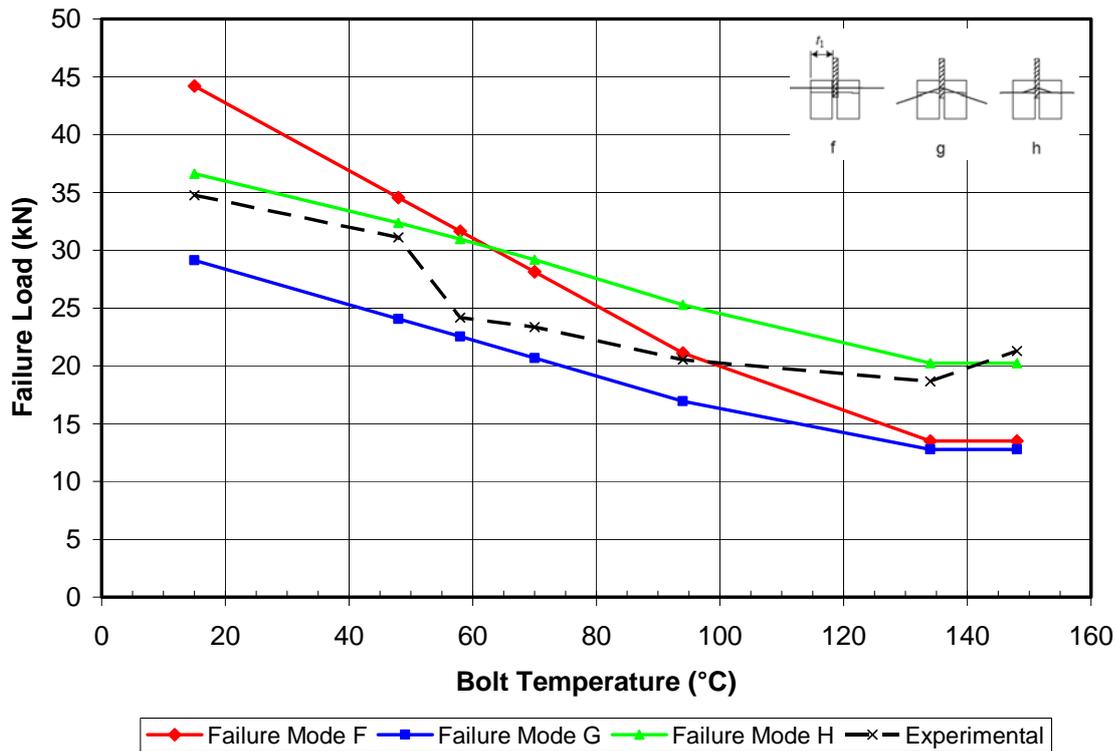


Figure 21 Predicted and experimental failure loads for single-bolt W-S-W connections tested at constant temperature.

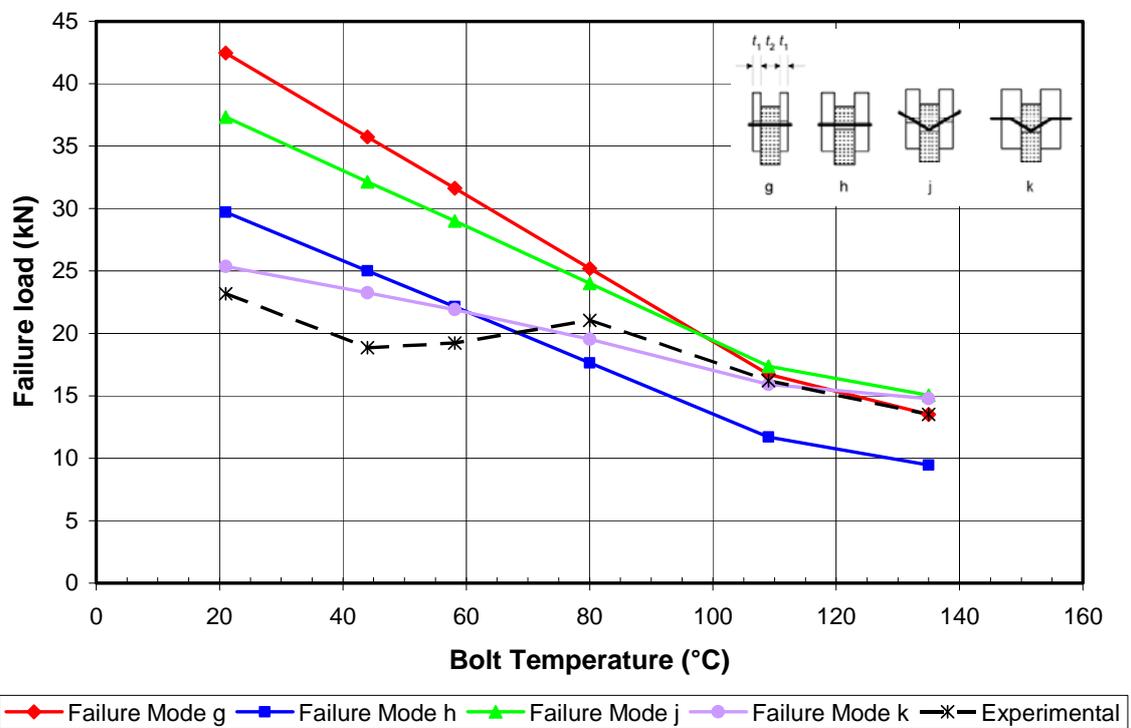


Figure 22 Predicted and experimental failure loads for single-bolt W-W-W connections tested at constant temperature.

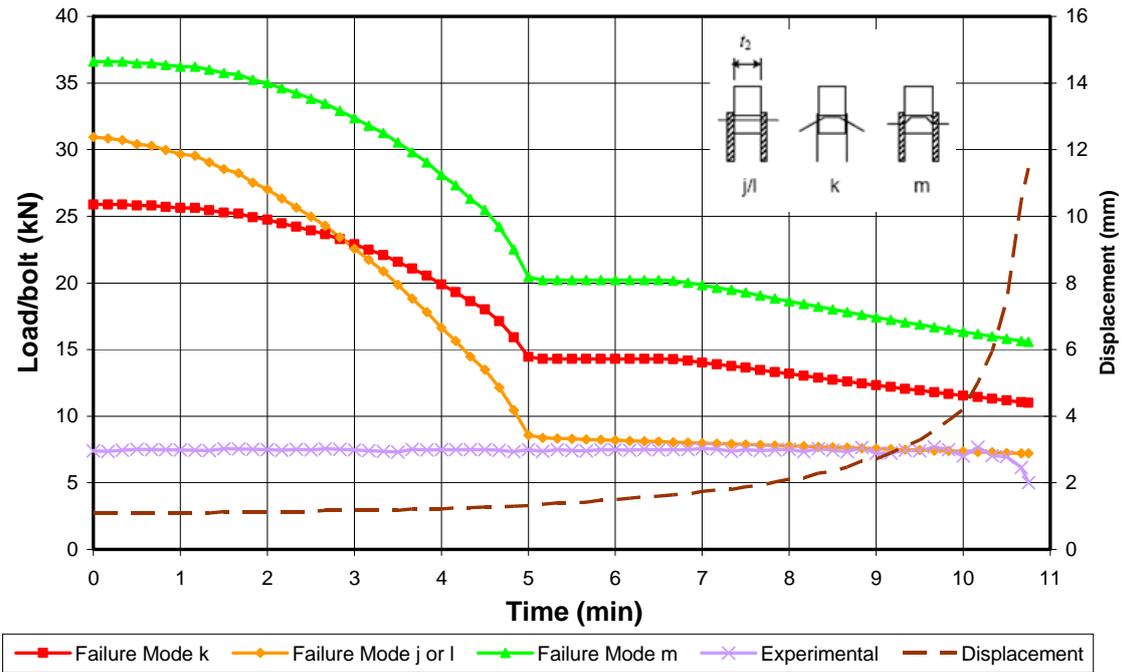


Figure 23 Predicted failure loads for S-W-S bolted connection

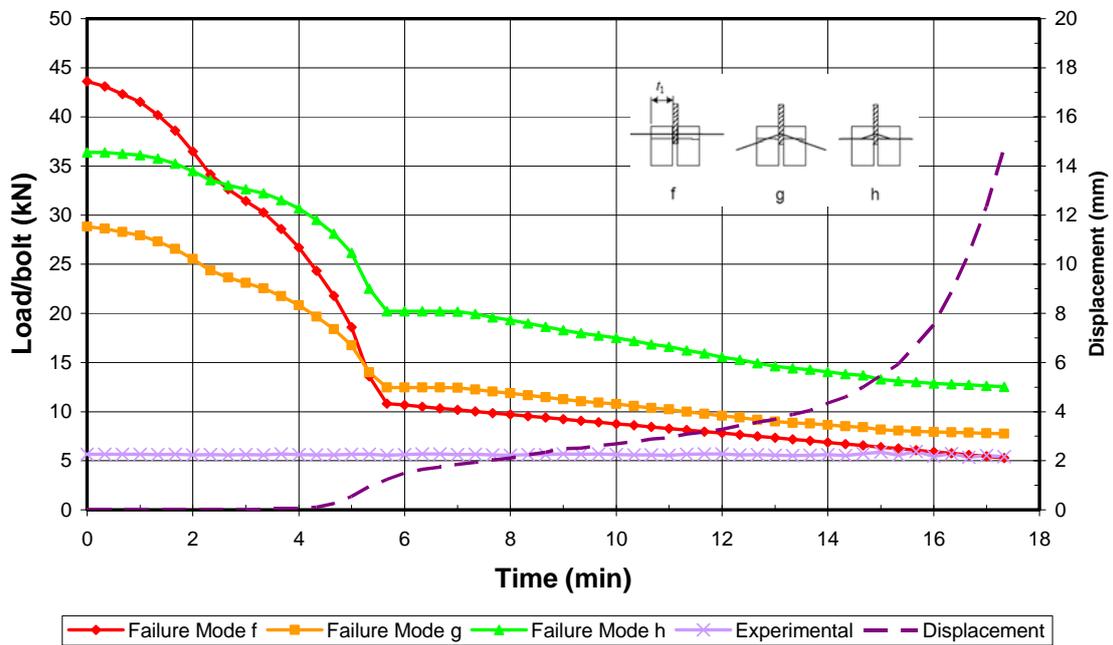


Figure 24 Predicted failure loads for W-S-W bolted connection

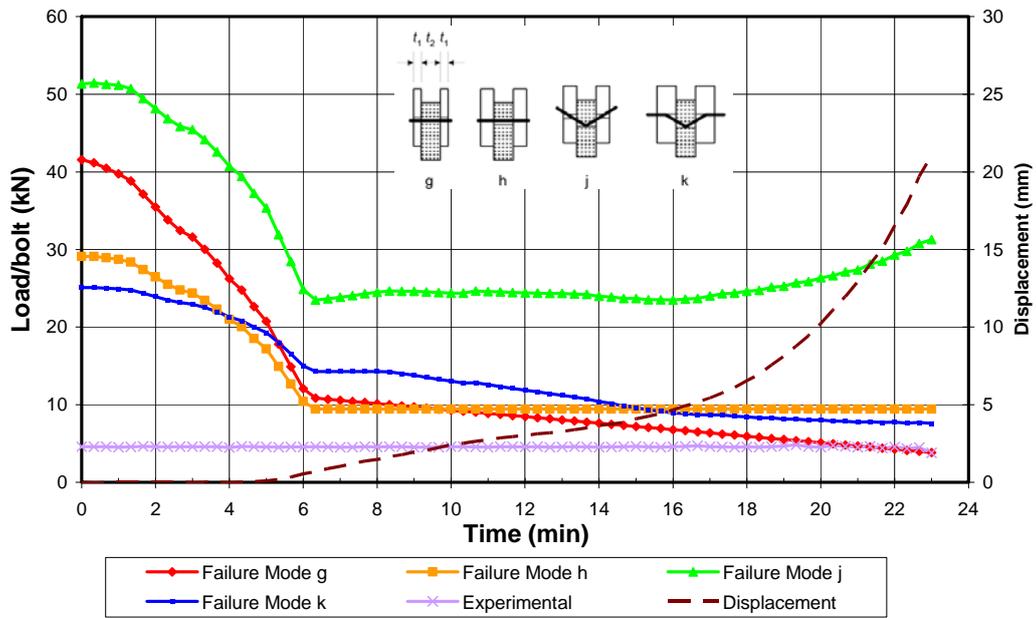


Figure 25 Predicted failure loads for W-W-W bolted connection

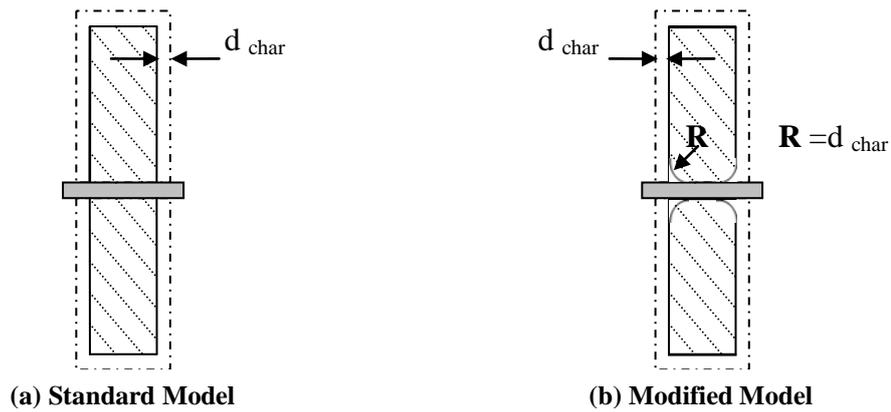


Figure 26 Schematic drawings of Standard Model and Modified Model.