# EXPERIMENTAL FIRE TESTING OF CONCEALED STEEL-GLULAM TIMBER SEMI-RIGID BOLTED CONNECTIONS

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**Abstract:** The outcomes of an experimental study aimed to investigate the fire behaviour of glulam beam-to-column semi-rigid connections exposed to standard fire while subjected to constant static load are presented in this paper. Each test assembly consisted of two glulam beam sections that were simply supported at their far ends and connected to an inversely-loaded glulam column in the middle using two concealed steel T-stub connectors. Two test variables: the bolt's end distance and the number of bolt rows, were investigated in four large-size glulam beam-column assemblies. The test assemblies were exposed to elevated temperatures followed the CAN/ULC-S101 standard time-temperature curve inside the large-size fire testing furnace accommodated in Lakehead University's new Fire Testing and Research Laboratory. Two draw-wire displacement sensors were installed outside the furnace then attached to the test assembly using ceramic rods to measure the vertical displacements of the left and right-side beams, as well as a third displacement sensor to measure the column vertical displacements. In addition, several thermocouples were implemented inside the glulam beam section to capture the elevated temperatures at various locations and depths underneath the gradually-developed char layer during fire tests. Once the failure criterion, which is mainly deflection dependent, was reached, test was terminated. Test results revealed that increasing the number of bolt rows from two to three rows, each of two bolts, resulted in an increased fire resistance time of a greater increment compared to that achieved by increasing the bolt's end distance from four to five-time bolt diameter.

## 1 INTRODUCTION

Glue-laminated timber (Glulam) beams and columns are composed of smaller sections of wood, known as lamina, that are glued together to form a final product that has a larger length and cross section than its individual components. Additionally, the mass timber sections developed using this manufacturing process have stronger mechanical properties compared to the individual lamina. This manufacturing process provides an advantageous approach for utilizing trees that were previously ignored as building materials due to their small size or low mechanical properties. In Canada, larger glulam members have generally been used in the construction of industrial, commercial, and institutional buildings for a few decades. However, recent changes to a few provincial building codes allowing for the use of wood as the primary building material in mid and high-rise construction, have increased the use of glulam sections in the Canadian building construction market. Since glulam structural members can be made to have any cross section, length, or have a curved shape, it is possible to manufacture heavy-timber components capable of resisting heavy loads. Unfortunately, current technical design documentation does not provide adequate design guidelines for heavy-timber connections having certain degree of moment resistance. While the Canadian Wood Design Handbook recommends a minimum end distance of four or five-time bolt diameter for connections resisting compressive or tensile axial loads, respectively (Canadian Wood Council 2015), no provisions are provided for a connection that would include both tensile and compressive forces, such

as in the case of semi-rigid beam-to-column connections. The primary structural concern in semi-rigid timber connections is the formation of brittle failure modes under loads; however, several studies have proposed that it is achievable to reduce or eliminate brittle failure modes in heavy-timber connections with an adequate end distance from the bolts (Hampson et al. 2003). Metallic connectors specifically, can promote ductile behaviour in heavy-timber connections by providing a source of energy dissipation (Murty et al. 2008 and Andreolli et al. 2011), or can promote brittle failure modes in the wood before the connection achieves a ductile failure mode (Humbert et al. 2014). According to recent studies that investigated the capacity of moment-resisting frames (Zarnani and Quenneville 2014) and their behaviour under lateral loading (Xiong et al. 2014), semi-rigid heavy-timber connections that posses certain degree of moment resistance will be increasingly required for mid and high-rise framed structures. In the eye of the public, the greatest concern with the increased use of wood as a primary construction material is the fire resistance of the final structure. While most current guidelines for fire design are prescriptive, the general trend has been to adopt a performance-based methodology for structural fire design (Hadjisophocleous and Benichou 1999). This trend has been supported by extensive testing of building materials as both individual components, sub-assemblies, assemblies and even entire structures; however, the focus has predominantly been on steel and concrete components, not glulam or other timber products (Bisby et al. 2013). Specifically, there is no available literature on the fire behaviour of heavy-timber moment-resisting connections either as individual components or as part of a structural assembly. The focus has primarily been on axially-loaded connections (Racher et al. 2010) under either tensile or compressive loading parallel to the grain (Peng. et al. 2010), or perpendicular (Audebert et al. 2012). These studies showed that timber connections mainly fail due to brittle failure modes in the wood, such as hole elongation, splitting, row shear-out, crushing, or due to yielding in the steel components at elevated temperatures. It was also noticed that concealed steel components in wood-steel-wood (WSW) connections were partially insulated from the effects of fire by the wood section(s) when compared to steel-wood-steel (SWS) connections, and benefit from a thermo-hydric transfer that occurs when water vapors escaping the wood condense on the steel components and increase the time increment at which the concealed steel components maintain a temperature of 100°C (Samake et al. 2014).

This paper presents the results of an experimental study undertaken to evaluate the behaviour of concealed steel-glulam beam-to-column semi-rigid connections at elevated temperatures. Four full-size beam-to-column test assemblies loaded to their full design capacity were exposed to a standard fire until failure occurred. Specimens consisted of double-cantilevered glulam beams connected with two concealed steel T-stub connectors bolted to a central glulam column. Test variables included bolt end distance and number of bolt rows in the connection.

## 2    Concealed Beam-to-Column Connections Testing Program

Based on previously documented failure modes of this connection type at normal temperature (Salem and Petrycki 2016), it was observed that an initial drop in the connection moment-resisting capacity occurred when row shear-out developed in the bottom row of bolts under tensile stresses. Therefore, the test assemblies were loaded so that the connection would undergo 100% of the designed row shear-out capacity as determined by Cl. 10.4.4.4 from the 2015 Canadian Wood Design Handbook (Equations 1, 2 and 3). The authors determined that a side member reduction factor was not necessary for the loading of the test assemblies presented in this paper as based on the background literature of the clause "slotted members did not experience as drastic of a reduction in row shear-out resistance as those WSW connections are composed of two separate timber members," (Mohammad and Quenneville 2001).

[1] $PR_{rT} = \Sigma(PR_{ri})$

[2] $PR_{ri} = \phi_w(PR_{ijmin})n_R K_d K_{sf} K_T$

Where; $\phi_w = 0.7$ (factor for brittle failure)

$n_R = 1.0 \ (No. \ of \ fastener \ rows), K_d K_{sf} K_T = 1.0 \ (Duration, Service \ and \ Treatment \ Factors)$

[3] $(PR_{ijmin}) = 1.2 f_v (K_{ls})(t) n_c (a_{ci})$

Where; $f_v$ = 2.5 MPa (Specified Shear Strength), $K_{ls}$ = 1.0 or 0.65 (Factor for member loaded surfaces)

$t$ = 62 mm (member thickness), $n_c$ = 2 (number of connectors in row), $a_{ci}$ = 3.5D or 4.5D = 66.0 mm or 88.0 mm (unloaded end distance)

## 2.1    MATERIALS

### 2.1.1    Glulam Beams and Columns

Beams and columns used in the test assemblies were composed of rectangular black spruce glulam sections sized 318 mm x 137 mm, manufactured to meet the 24F-ES/NPG stress grade and an architectural appearance grade. The individual lamina stock that were used to build up the sections measure 25 mm x 50 mm, were feather jointed at their ends and glued together in horizontal and vertical layers. The principle mechanical design properties of the glulam sections in the longitudinal direction are listed in Table 1.

Table 1: Mechanical properties of glulam beams

| Property | Strength (MPa) |
|---|---|
| Comp. parallel to grain | 33.0 |
| Comp. perpendicular to grain | 7.5 |
| Tension parallel to grain | 20.4 |
| Modulus of elasticity | 13100 |
| Modulus of rupture | 30.7 |

### 2.1.2    Steel T-stub Connectors

Beams were bolted to concealed T-stub connectors made of 12.7 mm- (1/2 inch) thick plates of 300W grade steel. The plates were secured to either side of the glulam column using 450 mm long fully-threaded steel rods with a diameter of 19.1 mm (3/4 inch) that pass through the column section. The concealed steel plates were connected to the glulam beams using 19.1 mm (3/4 inch) diameter A325M high strength steel bolts measuring 175 mm in length.

## 2.2    Test Assembly Details and Fabrication Process

Four full-size glulam frame connections with one wood-steel-wood connection configuration have been experimentally examined under standard fire exposure. Two assemblies have beam sections, each connected with four A325M bolts in two rows, while the other two assemblies used six bolts made of the same material and diameter in three rows to connect each beam section to the supporting column. For the two connection assemblies having the same number of bolts, two different bolt's end distances were used for the beams: four and five-time bolt diameter. Detailed sectional plans of the steel connectors having four and five-time bolt diameter end distances are shown in Figure 1. The bolt's end distance is defined as the distance between the beam end and the centre line of the first column of bolts.

Two steel T-stub connectors with the same configuration were used in each connection assembly. As illustrated in Figure 2a, the two T-stub connectors are attached to the glulam column using four long 19.1 mm (3/4 inch) diameter fully-threaded steel rods. This was facilitated by drilling four holes through the column to accommodate the steel rods. Bolt holes were punched into the steel connectors, and matching holes were drilled into the beam sections. Slotted cuts were made into the beam cross sections using a portable wood sawmill to accommodate the steel plates. Cylindrical notches were cut in the ends of the beams to accommodate the nuts attaching the steel connectors to the column. The notches were located above and below the rows of beams' connecting bolts and outside the potential pull-out path of those bolts. Figure 2b shows a general test assembly with four bolts in two rows in each beam section.
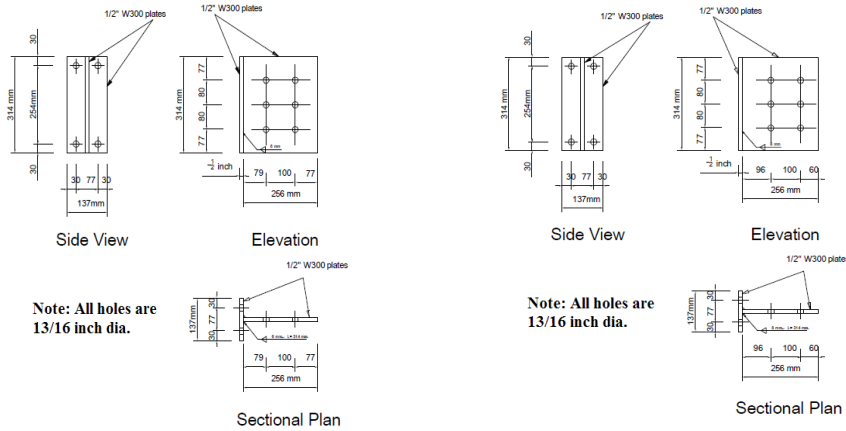
Figure 1: Steel T-stub connector details for four (left) and five-time (right) bolt diameter end distances



(a) Steel T-stub connectors

(b) A general beam-to-column test assembly

Figure 2: A general beam-to-column test assembly with four bolts in two rows.

Test designations and relevant details of the four test assemblies are summarized in Table 2 below.

Table 2: Elevated-Temperature Tests Matrix

| Test No. | End Distance | No. of Bolts |
|---|---|---|
| 1F | four-time bolt dia. | 4 |
| 2F | four-time bolt dia. | 6 |
| 3F | five-time bolt dia. | 4 |
| 4F | five-time bolt dia. | 6 |

## 2.3    Elevated-Temperature Tests Setup and Procedure

Each test assembly was inversely carried on two supports 3000 mm apart, while the entire assembly and supports were placed inside a large-size fire testing furnace accommodated at Lakehead University's Fire Testing and Research Laboratory, Figure 3a. The supports were secured to a strong bottom steel beam located underneath the furnace, which is part of the loading steel frame that is also supporting the furnace above the floor. Specimens were loaded to 100% of the calculated design capacity of the weakest connection. A powerful hydraulic jack installed and attached to the supporting steel frame above the furnace was utilized to apply loads on test assemblies, where the applied loads were transferred from the jack outside the furnace to the glulam column inside the furnace via an insulated steel post. Two draw-wire displacement sensors were installed outside the furnace and attached to ceramic rods that were inserted

into the furnace to capture the vertical displacements of the left and right-side beams. The ceramic rods were placed on the assembly 200 mm to left and right of the glulam column, Figure 3b. In addition, a third displacement sensor was attached to the top of the loaded glulam column to capture its vertical displacements during fire tests. The measured displacements from all three sensors were used to determine the rotations of the beam connections. Also, to measure the temperature of the wood and steel components of the connections during fire tests, twelve k-type thermocouples were placed on the specimen as detailed in Figure 3b. Six thermocouples were installed in the wood section at a depth of 30 mm, representing the midpoint of the wood sections in the transverse direction on either side (front and back) of the concealed steel plate. The other six thermocouples were used to measure the temperature of the steel connectors, four on the plate and two on the bolt heads to compare the internal and external steel temperatures during fire resistance tests.
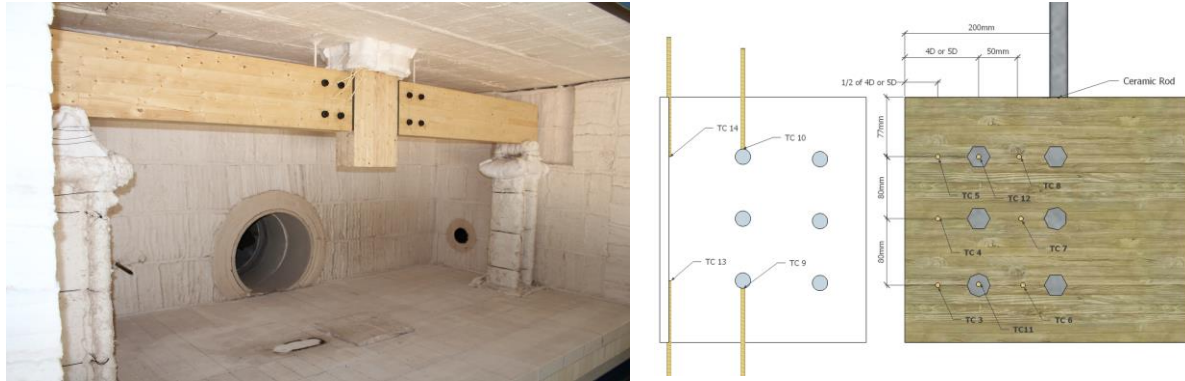


(a) A general test setup inside the fire testing furnace          (b) thermocouples layout

Figure 3: A general beam-to-column test assembly inside the furnace with all thermocouples and instrumentation installed.

As per CAN/CSA-S101, the total transverse load was applied in 25% increments at least 30 minutes before the test assembly was exposed to a standard fire, which was controlled and monitored by a built-in computer system and shielded thermocouples that all are integrated in the custom-designed fire testing furnace. A glulam beam-to-column connection having a five-time bolt diameter end distance and six bolts in three rows is shown undergoing exposure to a standard fire in Figure 4. Deflections of each test assembly were measured during fire testing, and once it was confirmed by measurements that the magnitude of deflection coincided with the previously-measured deflection at connection failure at normal temperatures, test was terminated. Afterwards, cross-sectional cuts were made into the charred samples to determine the charring rate of the glulam test assemblies during fire testing. In addition, the char layer was removed around the beam connections to investigate and then confirm the different failure modes.



Figure 4: Beam-column test assembly with six bolts in three rows exposed to a standard fire.

## 3    RESULTS AND DISCUSSION

The vertical displacements measured using the draw-wire displacement sensors were used to calculate the gradually increased rotations of the glulam beam-to-column connections as the assembly underwent exposure to a standard fire. Thermocouples TC3 to TC8 were used to determine the temperatures of the wood section halfway between the concealed steel plate and the external char layer, and were placed to be in the center of the wood sections resisting row shear-out in the compression, tension, and neutral-axis bolt rows. Thermocouples TC9 to TC14 measured the temperatures of the concealed plates at the tension and compression bolt row depths, as well as the temperatures of the bolt heads located in the tension and compression bolt rows. Rotations of both test assembly's sides were found to be in good agreement.

### 3.1    Time-Temperature Curves

Figures 5a to 5d illustrate the time-temperature curves of Tests 1F to 4F, respectively. Generally, it was noticed that the internal temperatures of the wood sections and the temperatures of the steel components were much lower than the environment temperature inside the furnace. It was also observed that rapid rotations began to occur in the connections when the temperatures of the wood section midway between the char layer and the concealed steel plate located in the lower bolts row resisting shear-out reached about 200°C, a state where the wood mechanical properties started to deteriorate rapidly prior to completely charred. The time-temperature curves developed in all fire tests were found to be in good agreement with each other. A hydrothermal relationship was observed to occur on the concealed plates in all four tests.
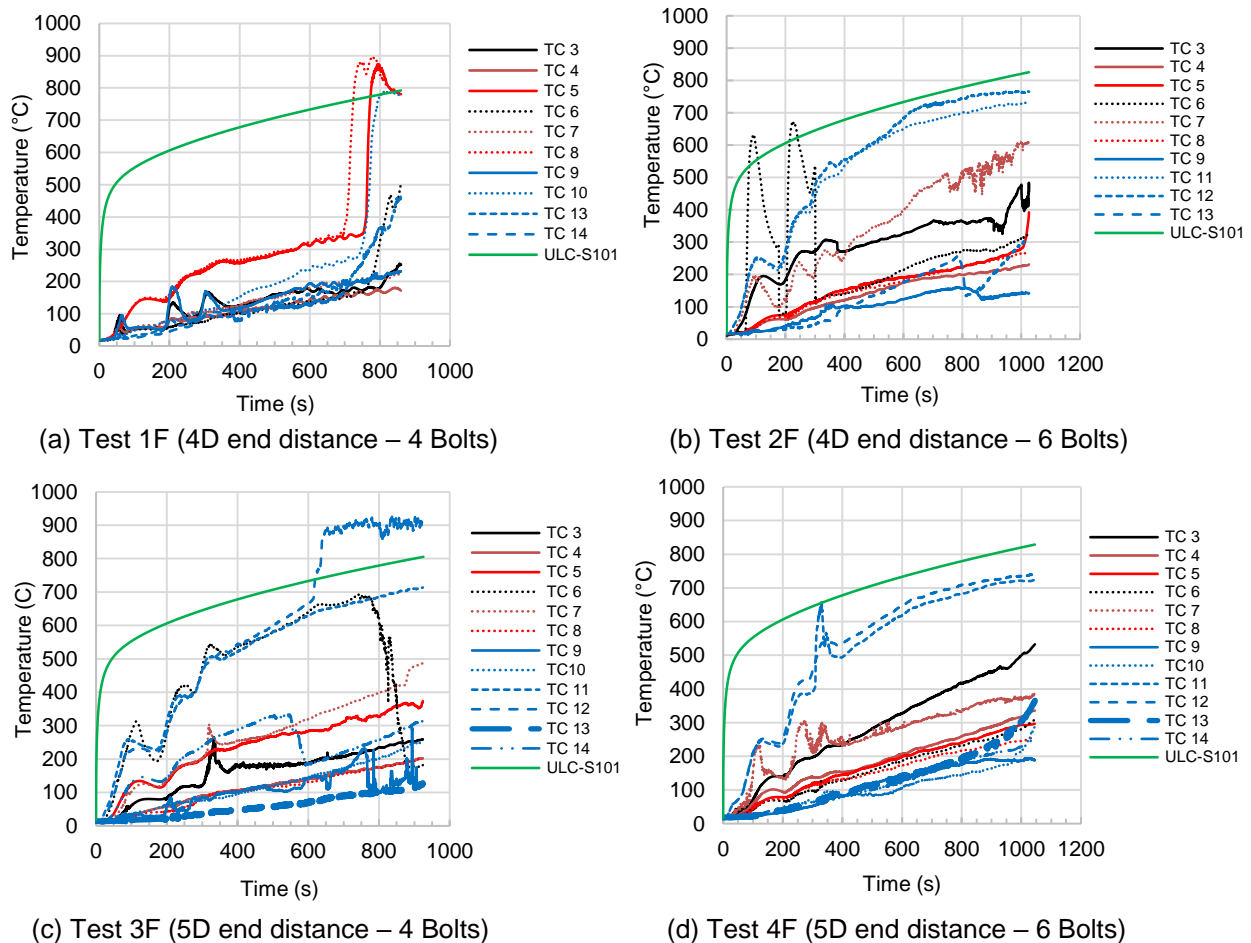


(a) Test 1F (4D end distance – 4 Bolts)

(b) Test 2F (4D end distance – 6 Bolts)

(c) Test 3F (5D end distance – 4 Bolts)

(d) Test 4F (5D end distance – 6 Bolts)

Figure 5: Actual time-temperature curves of all thermal measurements taken in fire resistance tests 1F through 4F

### 3.2    Effect of Bolt End Distance on the Connection Rotations and Fire Resistance Time

Generally, it was observed that the test assemblies underwent two different trends of increased rotations with time in all four fire tests. The connection rotations slightly increased in a linear trend during the initial test time (about 3 to 5 minutes); however, once enough time elapsed and the standard fire temperature elevated, the rotations increased exponentially over time until failure. As shown in Figure 6a, increasing the end distance from four to five-time bolt diameter in the four-bolt connections incrementally increased the connection fire resistance time by a slightly greater margin than that seen when increasing the end distance in the six-bolt connections, Figure 6b. For the four-bolt connections, increasing the end distance from four to five-time bolt diameter increased the fire resistance time by about 100 s. While for the six-bolt connections, increasing the end distance increased the fire resistance time by about 70 s. It was also observed that increasing the end distance did not change the rate at which rotations increased with time during failure, but it delayed the time at which more rapid failure would begin to occur in the connections.
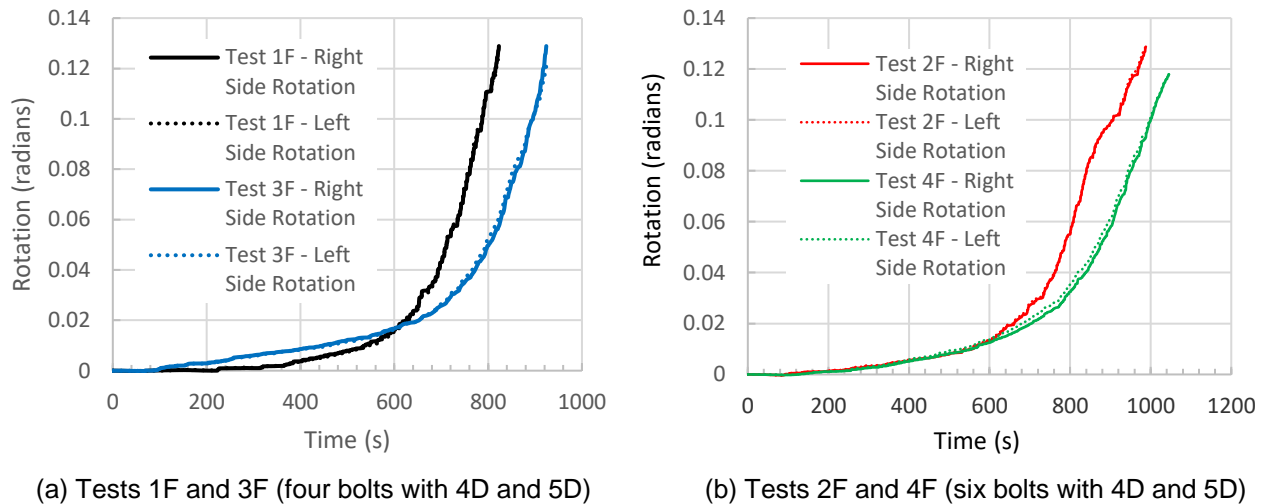
(a) Tests 1F and 3F (four bolts with 4D and 5D)    (b) Tests 2F and 4F (six bolts with 4D and 5D)

Figure 6: Time-rotation relationships developed to illustrate the effect of changing the bolt end distance in fire resistance tests 1F through 4F

### 4    Effect of Number of Bolts on the Connection Rotations and Fire Resistance Time

(a) Tests 1F and 2F (4D end distance, 4 and 6 bolts)   (b) Tests 3F and 4F (D end distance, 4 and 6 bolts)
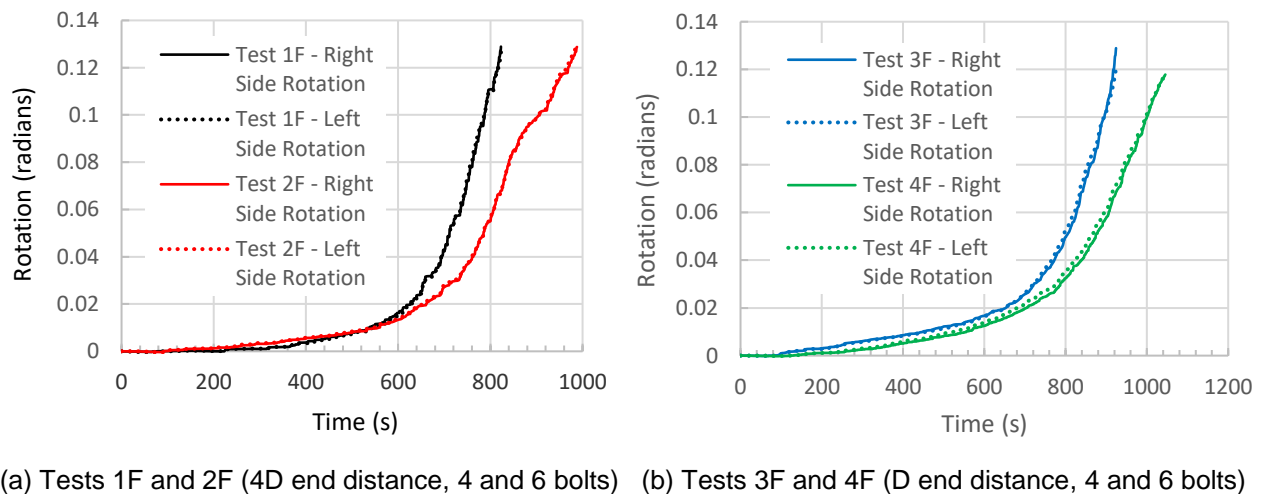
Figure 7: Time-rotation relationships developed to illustrate the effect of changing the number of bolts in fire resistance tests 1F through 4F

Figures 7a and 7b illustrate the effect of changing the number of bolts from four bolts in two rows to six bolts in three rows for four and five-time bolt diameter end distance connection assemblies on the connection rotations and fire resistance time, respectively. For the connections with four-time bolt diameter end distance, increasing the number of bolts from four to six increased the assembly fire resistance time by about 165 s; while in the five-time bolt diameter end distance connections, changing the number of bolts increased the assembly fire resistance time by about 134 s. The marginal increase in the fire performance of the different beam-to-column test assemblies was more pronounced in the connection with an end distance of four-time bolt diameter when compared to the results of the connection with an end distance of five-time bolt diameter. Additionally, it was observed that increasing the number of bolts from four to six, increased the stiffness of the connection, which is highlighted in Figures 7a and 7b, where the slope of the connection time-rotation curves for the six-bolt connections during failure at elevated temperatures is reduced when compared to the assemblies connected with four bolts in two rows.

## 4.3    Failure Modes

A few brittle failure modes, such as row shear-out and splitting, were observed in the glulam beams of all test assemblies. Figure 8a shows the burned glulam beam sections of Test 1F with four bolts and four-time bolt diameter end distance. The same figure highlights the hole elongation mostly presented in  the bolt holes  at the lower row exposed to tensile stresses. This suggests that the rapid increase in the connection rotations preceding failure from fire exposure was mainly caused by the high temperatures transferred in the steel bolts, and thus caused excessive charring of the wood around the bolt holes that led to hole elongation in a direction opposite to the tensile force direction at the connection bottom side and a rapid loss in the connection stiffness. Most interestingly, the inside of the slotted cuts was found to be thermally degraded but with no noticeable char layer formed nearby the concealed steel plates. In addition, the wood around the connections was found to have very little residual strength with several connections snapping apart by hand while removing them from the furnace and during cleaning after the fire resistance tests.



(a)  Burned glulam beam sections with the char layer removed

(b) Comparison between two similar connections with and without fire exposure

Figure 8: Row shear-out and splitting failure modes experienced in Test 1F assemblies with and without fire exposure

Moreover, the concealed steel connectors experienced minor thermal damage and minimum yielding in all four fire resistance tests; however, a few steel bolts experienced some thermal deformation and yielding in all test assemblies. The yielding of the connection steel components can be assumed to have occurred when the steel began to reach temperatures above 400°C, which coincided with a rapid increase in the connection rotations eventually in the fire resistance tests. For all connections, failure was first observed to occur along the line of the bottom row of bolts. Failure along this row resulted in splitting or shear pull-out of the wood between the end of the beam and the first bolt, then between the first bolt and second bolt in the row. This failure typically continued to propagate past the bolt farthest from the end of the beam as a split in the wood section propagated. This is a similar failure mode to that undergone by the connections loaded to failure under normal temperatures as illustrated by the comparison in Figure 8b. Also, it was

noticed that hole elongation occurred around the bolts farthest from the column-beam interface where the wood did not undergo brittle failure in the form of splitting, row shear-out, or crushing.

## 4.4 Summary of Test Results

In general, increasing the number of bolts increased the beam-to-column fire resistance time by a greater increment than that observed when increasing the end distance from four to five-time bolt diameter. Table 3 provides a summary of the four fire resistance tests' results.

Table 3: Summary of results of the four fire resistance tests on glulam beam-to-column assemblies

| Test No. | End Distance | No. of Bolts | Fire Resistance Time (seconds) | Fire Resistance Time (minutes) |
|---|---|---|---|---|
| 1F | 4D | 4 | 823 | 13.72 |
| 2F | 4D | 6 | 988 | 16.47 |
| 3F | 5D | 4 | 924 | 15.40 |
| 4F | 5D | 6 | 1058 | 17.63 |

## 5 CONCLUSIONS

Based on the experimental outcomes and the analysis of the fire resistance test results afterwards, a few conclusions have been driven, and are listed as follows;

1. Increasing the number of bolts from four bolts in two rows to six bolts in three rows increased the fire resistance time of the glulam beam-to-column connection by a greater increment than that observed by increasing the end distance from four to five-time bolt diameter;

2. The incremental increase in the fire resistance time from a greater end distance was marginally more effective in the connections with two rows of bolts in comparison to the connections using three rows of steel bolts, with two bolts in each row;

3. All test assemblies experienced substantial brittle failure modes, with minor deformations in the connecting steel bolts. Also, in cases of more pronounced brittle failure modes, the wood stress block resisting row shear-out was completely charred through;

4. The glulam beam-to-column connection's fire resistance was primarily governed by the shear resistance of the wood section resisting row pull-out driven by the force applied by the steel bolts through side bearing;

5. The primary concern for this wood-steel-wood connection configuration at elevated temperatures was shown to be the reduction in the cross-sectional area of the wood resisting row shear-out.

## 6 RECOMMENDATIONS

According to the experimental results obtained from this research project, it was seen that for concealed beam-to-column semi-rigid connections, increasing the number of bolts from four to six bolts increased the fire resistance time by a greater increment when compared to the fire resistance time added by increasing the bolt's end distance. Therefore, it is recommended that for the fire design of glulam semi-rigid connections six bolts to be used in three rows rather than four bolts in two rows. Either end distance of four or five-time bolt diameter is recommended to be used, as the incremental increase in the fire resistance time was very marginal; however, using a larger end distance will provide more conservative design. The scope of this research should be expanded to investigate the followings; the introduction of additional column of bolts, developing a T-stub connection that does not require circular notches to be cut in the glulam beam end, strengthening the connection against brittle failure modes in the wood sections, and applying reduced design loads until a fire resistance rating is achieved without failure in the wood sections. Glulam beams and columns framed using semi-rigid connections is a new application for the mass timber products that can be used as the primary construction material in tall buildings. Mid and high-rise buildings specifically are required to have semi-rigid connections with higher moment-resisting capabilities to rigidly sustain excessive lateral design loads, such as wind and earthquake loads. Moreover, having a sufficient fire resistance rating to safeguard people and property is another critical aspect in the design of tall timber

buildings. The design and construction of mid and high-rise buildings in Canada that are primarily made of timber is relatively being held back by the lack of well-developed design procedure for developing the required moment-resisting capacity of heavy-timber connections at both ambient and elevated temperatures.

## Acknowledgements

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