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SEISMIC PERFORMANCE OF BOLTED GLULAM TIMBER BRACE CONNECTIONS WITH INTERNAL STEEL PLATES

Zoe Baird[1], Joshua Woods[2], Christian Viau[3], and Ghasan Doudak[4]

ABSTRACT: Mass timber braced frame systems achieve their ductility through the brace connections. Canadian design standards currently lack guidance on how to detail bolted brace connections to achieve a target system-level ductility as defined in the National Building Code of Canada. The objective of this research is to develop guidelines on how to detail bolted glulam timber brace connections to achieve moderate or limited ductility. To accomplish this objective, a 4-storey prototype building was designed to determine realistic brace design forces as well as investigate how different parameters (e.g., fastener diameter and number of slotted-in plates) can impact the design of a timber braced frame. Based on the prototype structure, a connection with two internal steel plates was designed and detailed, which included consideration for the fastener slenderness and spacing to achieve ductile behaviour. To validate the performance of the proposed connection, full-scale testing under monotonic and cyclic loading was conducted. This paper discusses the results of the experimental testing, including connection stiffness, strength, ductility, as well as its energy dissipation capacity.

KEYWORDS: Mass timber, Braced frames, Connections, Ductility, Seismic loading, Experimental test

1 INTRODUCTION

Over the last 10 years, there has been a rapid increase in the number of mass timber buildings constructed in Canada [1], largely due to the environmental benefits of building with wood. Recent national and international climate change initiatives, including the *Canadian Net-Zero Emissions Accountability Act* and the *Paris Agreement*, strive to achieve net-zero emissions in the next three decades and emphasize that a key requirement in that effort is to increase carbon-neutral construction practices [2,3]. The International Panel on Climate Change (IPCC) recently published their list of strategies to reduce emissions by 2030 and "enhanced use of wood products" was one of 5 options in the buildings category [4]. Despite the environmental benefits of building with wood, questions remain surrounding the performance of tall wood structures, particularly under seismic loads.

Modern capacity-based seismic design guidelines allow engineers to detail specific structural elements or connections, commonly referred to as fuses, to dissipate energy during an earthquake in a ductile manner, while protecting brittle elements by designing them to higher seismic load levels through the use of overstrength factors. In the design of mass timber braced frames, steel-timber connections are typically relied upon to act as fuses, providing ductility and energy dissipation capabilities to the system. However, design and detailing considerations ensuring ductile behaviour and capacity-based protection against brittle failures of such connections are required to ensure adequate seismic performance, including the ability to achieve appropriate connection-level ductility to meet system-level demand.

The National Building Code of Canada (NBCC) recognizes both limited and moderate ductility braced frames for multi-storey mass timber structures up to 15 m and 20 m in height, respectively, in areas of high seismicity, and specifies respective system-level ductility modification factors (R_d) of 1.5 and 2.0 [5]. However, no guidance currently exists in the Canadian wood design standards (e.g., CSA O86) on how to design and detail the connections in braced frames to achieve the required system-level ductility [6]. This lack of design guidance can lead to inefficient design, excess material use, and inaccessibility of the structural system to designers [7].

There are several reported studies in the literature focused on bolted timber connections [8-14]. However, few studies have investigated axially-loaded timber brace connections at an appropriate scale for use in mid- and high-rise timber frames under cyclic loading. To address these gaps, the aim of this research is to develop connection design requirements for timber braced frames in taller timber structures, with the long-term goal of providing practical detailing requirements within the CSA O86 framework to achieve the system-level ductility modification factors in the NBCC. The specific objectives of this paper are to: (1) design a prototype mass timber braced frame structure in a Canadian region of moderate seismicity, (2) study the influence of connection design parameters, including bolt diameter, number of slotted-in plates, and bolt spacing, on the design strength and anticipated failure mode for the connection, and (3) evaluate experimentally the behaviour of a full-scale

[1] Zoe Baird, Queen's University, Department of Civil Engineering, Canada, zoe.baird@queensu.ca

[2] Joshua Woods, Queen's University, Department of Civil Engineering, Canada, joshua.woods@queensu.ca

[3] Christian Viau, Carleton University, Dept. of Civil and Env. Engineering, Canada, christian.viau@carleton.ca

[4] Ghasan Doudak, University of Ottawa, Department of Civil Engineering, Canada, gdoudak@uottawa.ca

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braced connection from the prototype structure under monotonic and cyclic loads.

2 BACKGROUND

There have been a number of studies that have focused on the behaviour of connections in mass timber structures, most of which involving small-scale connections (capacities ranging from 60 kN – 200 kN) [8-14]. These studies examined the influence of several connection design details, including dowel slenderness, fastener spacing, end distance [8,9], strengthening using self-tapping screws [10-12], effect of bolt pre-tension [13], and dowel material type, including shape-memory alloy [14]. These studies have considered a range of loading conditions to evaluate connection behaviour, including monotonic loads [8,11,12], cyclic loads [9,14,15], blast loads [16], and fire scenarios [17].

Beyond small-scale connection testing, there has been limited experimental research into the performance of large- to full-scale braced frame subassemblies or systems. Chen and Popovski [11] conducted testing on a multi-tier and single-storey braced timber frame with riveted connections, while Popovski et al. [18] conducted shake table testing on a single storey braced frame with bolted and riveted connections. These studies concluded that brace end connections can experience significantly different deformation levels, even if the two connection details are identical, and thus, the deformation capacity of a brace may not be equal to twice the capacity of a single connection [18]. A recent report by FPInnovations highlights the need for continued study into timber braced frames, particularly research on large-scale connections for multi-storey timber frames and their system-level behaviour [19].

2.1 DUCTILE DESIGN OF TIMBER FRAMES

Review of the literature has identified that there is a need to better understand the relationship between system-level behaviour of timber braced frames and the connection ductility demand, as well as the specific connection detailing needed to achieve the required system-level ductility. To address this concern, it is important to identify the hierarchy of ductility used in building design, which has been summarized by Gioncu [20]:

1. *Material ductility:* plastic deformation capacity of the materials (e.g., timber or steel);
2. *Cross-section ductility:* also referred to as curvature ductility, it is the plastic deformation capacity of the cross-section, considering interactions between elements of a cross-section (e.g., flange and web);
3. *Member ductility:* also referred to as rotation or displacement ductility and considers the properties of the member (e.g., length) and the formation of plastic hinges. In an axially loaded element, this could include deformation capacity of the connections; and
4. *System Ductility:* considers the displacement ductility of the entire structure under a lateral load distribution.

In this hierarchy, as one moves from material ductility to system ductility, the overall ductility decreases as it is often difficult to utilize all of the available ductility for complex structures (variability amongst connections and elements) under complex loading. Furthermore, this hierarchy was developed with steel or concrete structures in mind, in which the ductility of the system is typically achieved through yielding of the structural members (e.g., the brace in a steel braced frame or the beam in a reinforced concrete moment frame).

Capacity-based design of timber structures presents a unique challenge when compared with steel structures because of the brittle nature of timber in tension, which means that timber members are typically assumed to remain elastic while the connections are relied upon to act as fuses, providing ductility and energy dissipation capacity to the system. It has been shown by Chen and Popovski [7] that the system-level ductility of a multi-storey braced timber frame is a function of connection ductility, stiffness ratio between the connections and the brace, as well as the number of storeys in the structure. The system-level ductility of a timber frame, assuming all tiers yield simultaneously and that the structure behaves in the first mode, can be computed using Equation (1) [7]:

$$\mu_s = \frac{(\mu_{c1}+\mu_{c2})+k_r}{2+k_r} \tag{1}$$

where μ_{c1} and μ_{c2} are the ductility of the connections at either end of the brace, k_r is the ratio of the stiffness of the connection to the axial stiffness of the diagonal brace, equal to k_c/k_b where k_c is the stiffness of the connection and k_b is the stiffness of the brace.

While Equation (1) assumes that all storeys in a multi-storey braced frame yield simultaneously, this may be difficult to ensure in practice because of the uncertainty and variability surrounding connection capacity, as past tests have shown that there is the potential for only one connection in a brace to yield [7]. At the system-level, complex load distributions and the need to have several different connection designs over the height of a timber frame make it particularly challenging to ensure that all tiers in a braced frame will yield simultaneously. In such cases, a conservative approach to estimate system-level ductility would be to assume that only a single storey yields, and the system-level ductility can be computed using Equation (2) [7]:

$$\mu_s = \frac{(\mu_{c1}+\mu_{c2})-2}{m(2+k_r)} + 1 \tag{2}$$

where m is the number of storeys. To validate these proposed relationships, Chen and Popovski [11] conducted pushover analysis on five building archetypes while varying number of storeys, the ratio of the stiffness of the connection to the stiffness of the diagonal brace, and the connection ductility. In all cases, the braced frames had ductile riveted connections and the results showed that for a moderately ductile braced frame (R_d = 2.0) in which only one of the two brace end connections yield, a connection ductility of 11.5 is required. For situations in which both brace end connections yield, a minimum connection ductility of 6.3 is required. For limited ductility frames, connection ductility of 5.4 and 3.2 are required when one or both connections yield, respectively. An aim of the current research study is to

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investigate how to achieve these connection-level ductility requirements for bolted brace connections.

3 PROTOTYPE BUILDING DESIGN

A prototype 4-storey mass timber braced frame structure was designed for a site in Ottawa, Ontario, a region of moderate seismicity in Central Canada, where timber braced frames with moderate or limited ductility would be commonly utilized. The prototype structure was designed to determine realistic design forces and brace connection.

Figure 1 shows a plan and elevation view of the prototype structure, which includes 7 – 6 m bays in each orthogonal direction (42 m × 42 m total building footprint). The dead loads on the roof and floors were 1.88 kPa and 2.64 kPa, respectively, the snow load on the roof was 2.32 kPa for the building location, and the live load for a typical floor was 2.4 kPa for office occupancy, as per the NBCC [5].

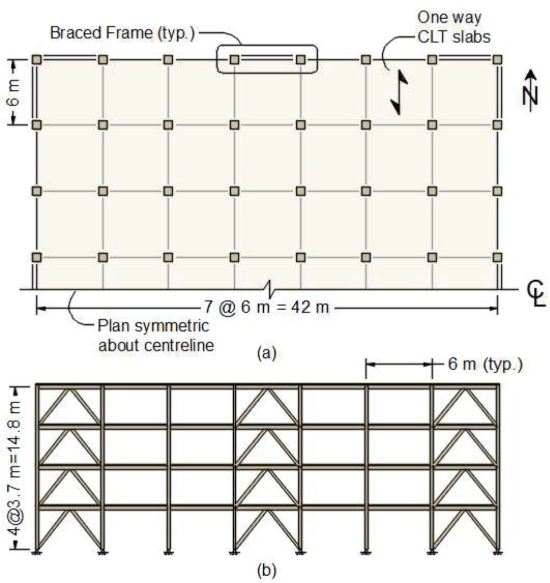

Figure 1: *Prototype structure: (a) plan view; (b) N-S elevation*

The gravity load resisting system in the prototype structure consisted of cross-laminated timber (CLT) slabs with glulam beams and columns. Based on the spans and loads, the selected CLT slabs were 5-layer (35 mm ply) 175 mm thick panels. The glulam beams and columns were grade 20f-E and 12c-E, respectively, and their respective sizes for each storey level are shown in Table 1. All structural members were sized according to the Canadian wood design standard (CSA O86) [6].

Seismic design of the prototype structure was carried out using the equivalent static force procedure outlined in the NBCC [5]. The base shear was determined for Ottawa (City Hall) and the building was assumed to be on a site designated as class C, corresponding to “firm ground”. The seismic weight was assumed to be 100% of the dead load plus 25% of the roof snow load. The fundamental period (T) used in the design of the prototype structure was assumed to be $T = 0.1N$, where N is the number of storeys in the structure [5].

Table 1: *Member selection summary (units in mm)*

	Storeys 1-2	Storeys 3-4
Brace	265 × 304	215 × 266
Interior Beam	265 × 456	
Exterior Beam	175 × 380	
Interior Column	265 × 304	
Exterior Column	265 × 266	

The lateral load resisting system for the prototype building consisted of mass timber braced frames in a chevron configuration in both orthogonal directions. For the frame design, the connections between the beams and columns were assumed to be pinned and the columns were continuous over the building height. Slotted-in steel plates were considered for the brace connections. Figure 2 shows an example of the brace connection with two slotted-in steel plates. This type of connection is appealing due to the high connection strength and stiffness while allowing the wood to protect the steel components against fire. This type of connection is used with dowel-type fasteners, such as bolts or tight fit pins. Bolts were used in this research over tight-fit pins because of the beneficial roping effect provided by bolts, which may lead to improved connection performance.

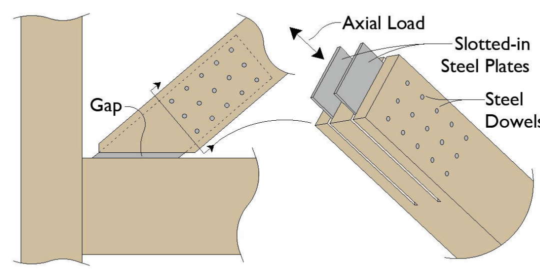

Figure 2: *Timber slotted-in steel plate connection*

One important consideration in the detailing of a timber braced frame is the need to leave a gap between the end of the brace member and the surrounding timber frame (illustrated in Figure 2). The gap allows the connections to deform and yield in both tension and compression, enabling the connection to achieve the required ductility and dissipate the maximum input energy. If no gap is provided, the brace element will be very stiff in compression, which would induce larger forces than expected under earthquake loads leading to less ductility and the potential for bending deformations in the columns which could lead to potential safety concerns [19].

3.1 CONNECTION DESIGN

The brace design forces were determined using the results of the equivalent static force procedure in the NBCC [5]. The resulting brace design force at the first storey of the prototype structure was approximately 250 kN, which was the target design force for the connection. To design the connection, five different failure mechanisms were considered: (1) net tension, (2) row shear, (3) group tear-out, and (4) fastener yielding. Figure 3 illustrates these failure modes.

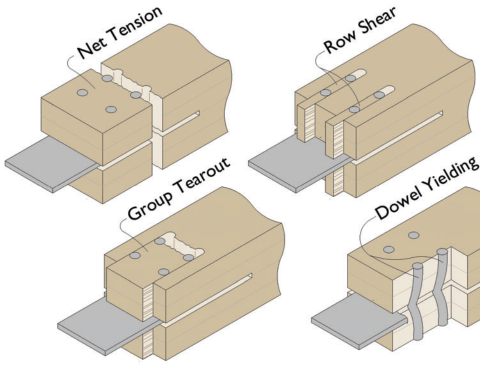

Figure 3: *Connection failure modes*

To initiate the connection design process, the number of fasteners required to achieve a yielding capacity approximately equal to the brace design force was determined, resulting in a 265 mm × 304 mm cross section being chosen. Two internal steel-plates with 10 mm thickness were used, which resulted in timber side member thicknesses of 71 mm [(304 – 2×10)/ 4 = 71 mm]. The fastener diameter was determined by setting a target slenderness ratio of 10 or higher for the connection, which is in line with recommendations from Eurocode 8 [21], which suggests fasteners with a diameter of less than 12.7 mm (1/2”) and a slenderness ratio greater than 10 be used for ductile timber connections. The bolts were grade ASTM A307 steel, with a specified yield strength (F_y) of 310 MPa. The yield resistance of the connection was determined according to the provisions of CSA O86 [6]. Ultimately, it was determined that 16 - 9.53 mm (3/8”) diameters fasteners were required to carry the factored load, which resulted in a yield capacity of 286 kN.

The capacity of the brittle failure modes (net tension, row shear, and group tear-out) were determined. In the initial design iteration, the minimum fastener spacing according to the Canadian design standard (CSA O86-19) of 4 times the fastener diameter (resulting in a minimum spacing of 38 mm) was used to layout the bolts. Based on this fastener spacing, the resulting row shear, group tear-out, and net tension resistances of the connection were 362 kN, 516 kN, and 1054 kN respectively.

While use of the minimum fastener spacing was satisfactory from a design perspective, it was anticipated that the actual yield resistance of the connection would be higher than the design yield resistance based on the code calculations because of higher yield strength of the bolts. Additionally, the potential for post-yield hardening behaviour in the connection response, could result in premature brittle failure. Consequently, a target minimum overstrength factor of at least 2, computed as the ratio of the resistance of the most critical brittle failure mode (e.g., lowest of the row shear, group tear-out, and net tension resistances) to the yield resistance of the connection was desired. To accomplish this objective, the bolt spacing was increased. Because net tension resistance was found to not govern the brittle failure mode, both row shear and group tear-out resistances could be increased by increasing the fastener spacing. Ultimately, a fastener spacing of 100 mm, which is more than twice the minimum fastener spacing specified in the Canadian wood design standard was selected, which resulted in row shear and group tear-out resistances of 905 kN and 652 kN, respectively, corresponding to an overstrength factor of approximately 2.3.

Finally, compression (i.e., buckling) and gross tension failures were considered in the brace member, which had capacities of 1162 kN and 1054 kN, respectively. Figure 4 shows the connection design proposed for the 4-storey prototype building.

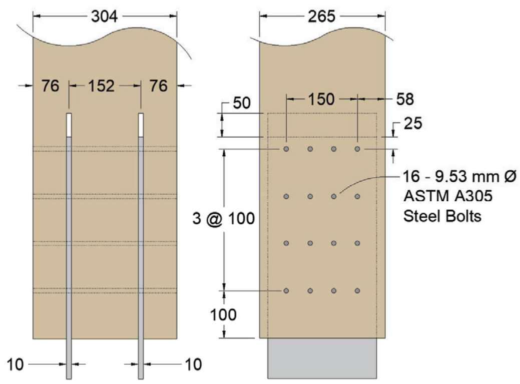

Figure 4: *Prototype connection design*

4 EXPERIMENTAL PROGRAM

To evaluate the ductility of the proposed connection design shown in Figure 4, full-scale connection tests were conducted. The connections were fabricated using grade 24f-E spruce-pine-fir (SPF) glued-laminated (glulam) timber supplied by Nordic Structures (Québec, Canada). The glulam had a density of 560 kg/m^3, a bending strength of 30.7 MPa, a longitudinal shear strength of 2.5 MPa, a compression perpendicular to grain strength of 7.5 MPa, and a modulus of elasticity of 13.1 GPa according to the manufacturer [22]. The bolts were grade ASTM A307, and from testing were found to have an average yield stress of 400 MPa (standard deviation = 16 MPa).

Figure 5 shows a typical specimen in the testing frame. Each specimen included two identical connections (details illustrated in Figure 4), with an overall length of 1350 mm to fit into the experimental testing frame. The holes in the timber were 10.7 mm (27/64”) in diameter, which is approximately 1.2 mm larger than the diameter of the bolts. The holes in the steel plates were 12.7 mm (1/2”) in diameter to allow for erection tolerance. The results of two specimens are presented in this paper. The first specimen, hereafter referred to as specimen M1, was tested under monotonic load, while the second specimen, referred to as specimen C1, was tested under simulated seismic load using a cyclic loading protocol.

4.1 EXPERIMENTAL TEST SETUP

The connection was tested in a frame with a capacity of 1350 kN in tension and the actuator had a stroke of +/- 250 mm. Figure 5 shows a typical test setup for the connection, which was used for both the monotonic and

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cyclic tests. Lateral bracing was installed at the top of member to prevent any out-of-plane deformation during testing. The slotted-in steel plates were attached to the actuator at one end and to a steel assembly at the other end. The steel assembly was fixed to the laboratory strong floor using 8 – 31.8 mm (1-1/4 in.) steel grade B7 rods.

A load cell mounted to the hydraulic actuator was used to measure the force and 12 potentiometers were used to measure the displacement response of the connections, which included two string potentiometers on each connection (top and bottom) to measure their individual displacement response. Linear potentiometers were also mounted across the slots in the timber for the steel plates to measure any potential opening during testing.

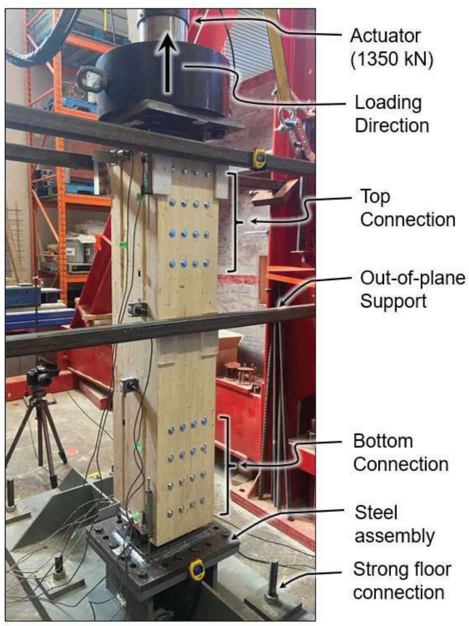

Figure 5: *Test specimens and experimental test setup*

4.2 LOADING PROTOCOL

Monotonic and cyclic loading tests were conducted based on the European Test Standard EN12512 – *Cyclic Testing of Joints made with Mechanical Fasteners* [23]. Monotonic testing was conducted to evaluate the connection strength and failure mechanism, as well as to determine the connection yield displacement (Δ_y) to be used in defining the cyclic loading protocol. The monotonic tensile test was conducted at a constant rate of 2 mm/min, which resulted in a total test time of approximately 20 minutes.

Based on the results of the monotonic test, a cyclic loading protocol was developed to study the connection behaviour under simulated earthquake loads. Figure 6 shows the cyclic loading protocol used in this study. The cyclic tests were executed in displacement control based on increments of the Δ_y of the connection determined from the monotonic test, which included displacement levels of 0.2, 0.4, 0.6, 0.8, 1.0, 1.5, and 2.0 times Δ_y, followed by cycles at increments of Δ_y to failure.

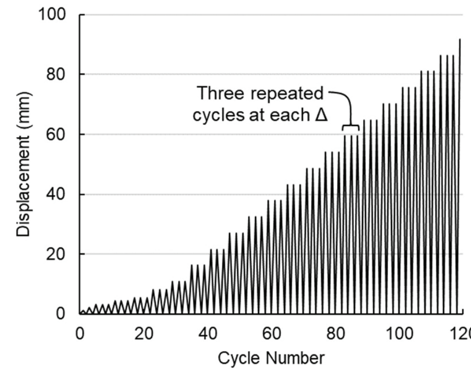

Figure 6: *Tension cyclic loading protocol* [22]

5 EXPERIMENTAL RESULTS

Table 2 summarizes the key structural response parameters for the specimens described in this paper, including the yield load (P_y), peak load (P_{peak}), and ultimate load (P_u) as well as the yield displacement (Δ_y), peak displacement (Δ_{peak}), and ultimate displacement (Δ_u). The yield load and displacement were determined using the method proposed by Yasumura and Kawai [24], which has been shown to produce consistent results for a range of force-displacement behaviours in the past [25]. The peak load was defined as the maximum load achieved during the test. The ultimate load and displacement were defined as the point when the load dropped by 20% from the peak load.

Table 2: *Connection structural response parameters*

	Specimen M1		Specimen C1	
Parameter	Top	Bottom	Top	Bottom
P_y (kN)	557	550	506	506
Δ_y (mm)	3.4	2.4	2.7	3.2
P_{peak} (kN)	1024	1024	775	775
Δ_{peak} (mm)	20.6	16.5	21.3	22.1
P_u (kN)	819	819	620	620
Δ_u (mm)	24.2	16.6	21.4	22.7

5.1 MONOTONIC TEST RESULTS

Figure shows the force-displacement response of the specimen tested under the monotonic load. The results for the top connection show a gradual increase in the load at small displacement levels, which corresponds to the closing of any gaps between the steel plates, timber, and the bolts, which are required for constructability of the connection. After closing any gaps, both top and bottom connections have an elastic behaviour followed by a gradual softening, which results from yielding of the bolts. Following bolt yielding, the connection exhibits post-yield hardening behaviour, which is attributed to embedment stiffness of the timber, which interacts with the bolts even after significant yielding of the fasteners. At a displacement of approximately 23 mm, several bolts in the top connection of the specimen fractured, resulting in a significant drop in load carrying capacity of the specimen.

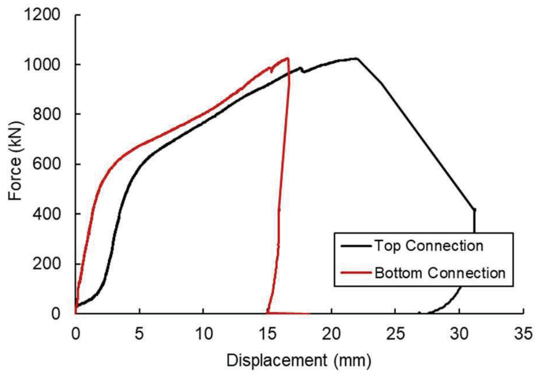

Figure 7: Connection M1 force-displacement response

Figure shows sample bolt deformation patterns from the monotonic test and the results show that bolts from both the top and bottom connections had significant plastic deformations and the bolts form six plastic hinges at the locations of the slotted-in plates

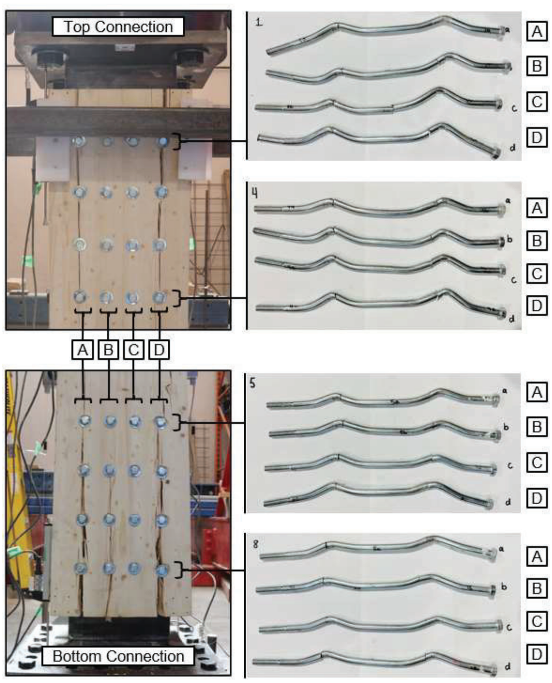

Figure 8: Connection M1 bolt deformation patterns

5.2 CYCLIC TEST RESULTS

Figure 9 shows the hysteretic force-displacement response of the bottom and top connections for the second specimen. The results show that the cyclic backbone curve was similar to that of the monotonic test, characterized by an initial gradual increase in stiffness as the bolts engage with the slotted-in plates followed by an elastic and post-yield hardening behaviour up to failure, which occurred due to excessive splitting of the wood. The hysteretic behaviour of the connection shows a highly pinched response, with stiffness degradation under repeated load cycles. This behaviour is attributed to crushing of the wood surrounding the dowel during the initial cycle and on the second cycle the dowel must close a gap before re-engaging the wood, at which point the stiffness of the connection increases. Both connections yield and achieve ductility of 7.1 and 8.1, which is attributed to the post-yield hardening behaviour of the connections. This level of ductility is within the required range suggested by Chen and Popovski [7] to meet the system-level ductility requirements for a moderately ductile timber braced frame according to the NBCC [5].

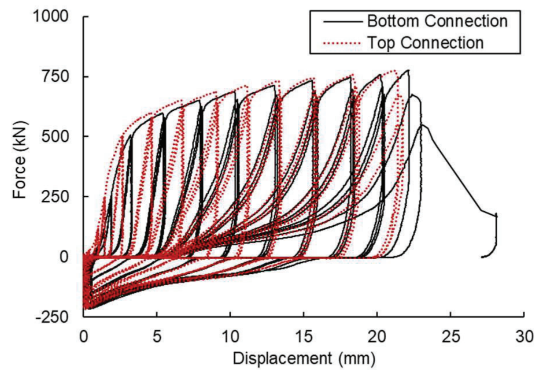

Figure 9: Connections C1 force-displacement response

Figure 10 shows the progression of splitting in the connection at various displacement levels throughout the cyclic loading protocol. The results show that no splitting occurred up to a displacement of approximately 13 mm. However, even after the onset of splitting, it did not result in any significant strength degradation and the connection maintained similar hardening behaviour up to failure.

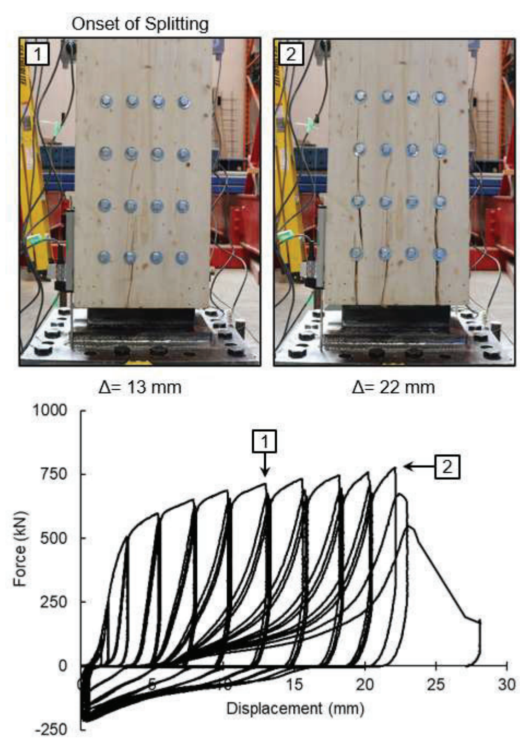

Figure 10: Specimen C1 bottom connection crack progression

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6 CONCLUSIONS AND FUTURE WORK

The use of mass timber braced frames is rising in Canada, but knowledge gaps remain on how to design timber connections to achieve system-level ductility levels in the NBCC. Through experimental testing of a connection based on the design of a 4-storey prototype structure, the conclusions of this study are as follows:

1. Bolted glulam timber connections with slotted-in steel plates can meet the strength requirements for multi-storey mass timber structures.
2. Emphasis should be placed on the ratio of ductile to brittle failure modes during the connection design phase. In this paper, connections tested with a capacity ratio of 2.3 resulted in high ductility.
3. Connection-level ductility requirements can be achieved to meet the system-level ductility demand for moderate and limited ductility braced timber frames.

The work presented in this paper is part of a larger study focused on the behaviour of bolted glulam timber connections with slotted-in steel plates. Future work includes experimental testing of connections with varying fastener diameter, bolt spacing, and number of slotted-in steel plates. The goal is to better understand the key factors that contribute to the design of a ductile timber connections so that design guidance can be provided in CSA O86 on how to detail timber connections to achieve the required system-level ductility levels in the NBCC.

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