Deconstructable Hybrid Connections for the Next Generation of Prefabricated Mass Timber Buildings

Final Report

This final report reflects the activities carried out from April 2020 to March 2021

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April 2021

ABSTRACT

Timber has been used for building construction for centuries, until the industrial revolution, when it was often replaced by steel and concrete or confined to low-rise housings. In the last thirty years however, thanks to the development of mass timber products and new global interest in sustainability, timber has begun to make a resurgence in the building industry. As building codes and public perception continues to change, the demand for taller and higher-performance timber buildings will only grow. Thus, a need exists for new construction technology appropriate for taller mass timber construction, as well as for fabrication and deconstruction practices that respect wood's inherent sustainable nature. With this in mind, this research program aims to develop a new hybrid shear connection for mass timber buildings that allows for easy construction, deconstruction, and reuse of the structural elements.

This report includes results of Phase 1, which focused on connections consisting of partially-threaded 20M and 24M steel rods bonded into pockets formed in CLT and surrounded by thick crowns of high-strength three-component epoxy-based grout. A total of 168 specimens were designed and fabricated, and push-out shear tests carried out with a displacement-controlled monotonic loading protocol. Strength and stiffness values were assessed and effective failure modes in specimens identified. These latter, along with the recorded load-deformation curves, indicate that it is possible to develop mechanics-based design models and design formulas akin to those already used for typical dowel-type fastener timber connections. Additionally, the specimens were easily fabricated in the lab and quickly fastened to the test jig by means of nuts and washers, suggested such connections have a strong potential for prefabrication, disassembly, and reuse.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the assistance of Brandon Chan, Pablo Chung, and Lief Eriksen of the Centre for Advanced Wood Processing, George Lee and Chao (Tom) Zhang of the Timber Engineering and Advanced Mechanics lab, and Blériot Vincent Feujofack Kemda of the Sustainable Engineered Structural Solutions lab at the University of British Columbia (Vancouver). The authors' thanks are also extended to Rothoblaas Canada (Delta, BC) for donating the threaded steel rods, nuts and washers, Hilti Canada (Langley, BC) for discounting the epoxy grout and donating the mixing paddle, Structurlam (Penticton, BC) for discounting the CLT panels, and Select Steel (Delta, BC) for including for free the priming of the test jig.

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1 INTRODUCTION

In 2015, the National Building Code of Canada (NBC) adopted six-storey mass timber construction. This, along with a similar change to the American International Building Code (IBC), lead to an estimated eight-fold increase in the manufacturing volume of mass timber products in North America, from roughly 100,000 m^3 in 2015 to 820,000 m^3 in 2020 (Anderson et al. 2020). Canada saw the establishment and expansion of several mass timber manufacturing facilities in provinces like British Columbia, Ontario, and Quebec. The 2020 NBC has further adapted height limitation on mass timber buildings, moving from the previous six storeys height limit to twelve storeys. The adoption of this change by provincial building codes and municipal bylaws in the coming years, along with an increased national production capacity, will place unprecedented demand on the Canadian timber building industry and usher in a new era of tall timber construction in Canada. With this comes new limitations and new challenges to Canada's timber industry.

First, with the rise in timber construction, the impact on Canada's forests and the end-of-life options for mass timber products become important issues. Canada, though a global leader in responsible forest management, is one of the world's largest producers of construction waste (Earle, Ergun, and Gorgolewski 2014). Timber, touted for its ability to sequester carbon, will only do so as long as it is in use, eventually releasing its stored carbon back into the atmosphere upon demolition and landfilling. There is therefore a need for new mass timber construction technologies that are conducive to deconstruction and reuse, alleviating the demand on the forestry sector and reducing greenhouse gas emissions from the decomposition of demolished timber.

Second, from a design perspective, the growth in mass timber construction means a new need for structural systems and assemblies to be developed. The 2020 NBC dictates that only a percentage of a structure can consist of exposed timber. This leaves designers with two options: (i) all-timber construction with most of the structure and members covered in fire-retardant cladding, severely reducing the economic, environmental, and aesthetic advantages of mass timber construction, or (ii) hybrid construction, consisting of timber mixed with steel or concrete. The latter is not only more efficient, but also facilitate design compliance with the code's performance-based seismic and serviceability requirements. In tandem with the development of hybrid timber construction technology is a need for connections suitable for this type of construction, as well as for the new heights timber is now allowed to reach. Current mechanical fasteners such as nails, screws, truss-plates, and complementary steel hardware are suited for mid-rise timber construction. To continue using such connections for the greater loads and induced deformation on multi-storey all-timber or hybrid timber-based buildings would be both unsuitable and uneconomical. Additionally, the damage caused by the removal of such fasteners severely limits the potential for reuse of timber elements. As a result, there is a need for research on hybrid timber-based connections and assemblies that have the potential to be disassembled and reused.

This project represents Phase 1 of a multi-year research program aiming to develop and study a steel-to-timber shear connector that responds to the need for robust and deconstructable connection solutions for the next generation of hybrid mass timber buildings. Connections are conceived in order to (i) meet multi-objective structural performance of NBC and CSA-O86 standard, (ii) favour modular construction, (iii) favour quick erection of buildings, (iv) quick disassembly and possible reuse of the timber members, and (v) provide seismic-resistant structural assemblies.

Within this report, outcomes of Phase 1 are included, with specific attention to the fabrication and testing program performed on shear connectors made with 20M and 24M fully-threaded steel rods, CLT panels, and an epoxy-based grout reinforcement layer.

2 DESCRIPTION OF TESTED CONNECTIONS

The connection type studied in this project consists of a full-threaded (FT) steel rod embedded into cross-laminated timber (CLT) through a thick layer of epoxy-based grout mixture. The FT rod's arrangement allows components of timber assemblies to be mounted quickly and individual members to be fastened on-site using only nuts and washers. The grout layer acts as reinforcement, stiffening the rod and augmenting the loaded area, thus reducing the induced local wood embedment stress. In order to be considered fully deconstructable, the connection shall not yield under design loads so that no permanent plastic deformation occurs. Therefore, this connection is intended to be capacity-protected and designed to remain elastic under forces and displacement demands as defined in the CSA-O86 for non-dissipative connections. Connections are conceived to perform as laterally loaded shear connectors, although axial forces can also be transferred through the rod's axis. Shear connections are structurally versatile, with the potential to form hybrid timber-based floor diaphragms (Figure 1a) and shear wall assemblies (Figure 1b) of a building's lateral-load resisting system.

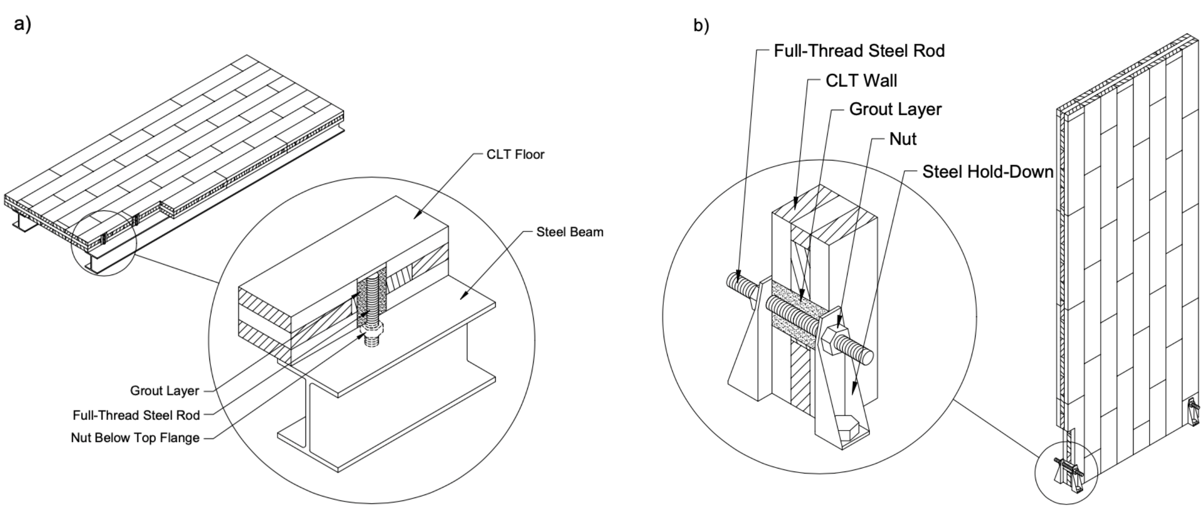

Figure 1: Application of shear connector in steel-timber (a) floor diaphragms and (b) shear wall systems.

3 Literature Review

Research on laterally loaded fastener connections in timber construction dates back to 1932, when Trayer (1932) conducted embedment strength tests on dowel-type wood connections. The results of this work were later modeled by Johansen (1949), resulting in the Johansen yield model for determining the capacity of dowel-type connectors in timber. The Johansen model was later modified by Larsen (1973), leading to the European Yield Model, which, courtesy to further research conducted by Soltis et. al. (1986), was included in the Eurocode 5 Part 1-1(EN 1995-1-1) – *Design of timber structures: Common rules and rules for buildings*. The Canadian standard CSA-O86 – *Engineering design in wood* makes use of similar equations, as presented by Whale et. al. (1987). It was not until an extensive research project beginning in 2006 lead by German researchers (Uibel and Blaß 2006) that these equations were expanded to include dowel-type connections in CLT.

Adhesive-reinforced dowel-type connections, known as a glued-in-rods, were studied in the late 1980s in two foundational research programs conducted in Scandinavia and New Zealand by Riberholt (1986) and Townsend and Buchanan (1990), respectively. Since then, glued-in-rods have been applied primarily in Europe in heavy timber column-to-foundation joints, moment-resisting joints, and as shear connectors for composite timber-concrete flooring systems. Despite continued research such as that conducted by Rodd et. al. (1989), Davis and Claisse (2001), Bainbridge et. al. (2002), and a three-year European-wide research campaign (Bengtsson and Johansson 2002), glued-in-rods have yet to be codified in the Eurocode, or any other major building code.

The first research program on glued-in-rods for CLT application was conducted in 2016 by Master's students at Aarhus University (Andersen and Høier 2016), testing 12 samples for an assessment of their axial pull-out strength capacity. A more comprehensive research campaign was later presented by Azinović et. al. (2018).

From a structural assembly point of view, Loss et. al. (2016) showed an urgent need for developeing new shear connections suited for hybrid CLT-based floor systems in tall timber building applications. Within this project, research is on the development of high-performance connections for mass timber buildings. Specifically, a novel hybrid timber-based shear connector is studied, which combines glued-in-rod technology with CLT panels. Such connections have only just begun to be studied, primarily in Europe and only for their axial pull-out capacity. The application of this type of connection as a shear connector in floor diaphragms and shearwalls of lateral-load resisting systems is completely novel. Furthermore, this is the first research program to conduct tests on such connections using Canadian products and with an emphasis on the potential for prefabrication, disassembly, and reuse.

4 MATERIALS AND METHODS

4.1 Materials

Materials used in this project are summarized in Table 1. A total of 24 3-ply 105 mm thick CLT panels were procured from Structurlam (Penticton, BC). Half of these were V2M1.1 visually graded panels, consisting of three layers of 35 mm thick SPF #2 boards, or better grades, and half were E1M5 electronically graded panels, consisting of three layers of 35 mm thick boards, of which the interior minor layer is SPF #2 or better grades, and the two face layers are MSR 2100 1.8E SPF. The panels were certified to meet the requirements of the Standard for Performance Rated CLT ANSI/APA PRG 320 and the APA Product Report PR-L314. An average moisture content of 9.3% was determined for the specimens based on measurements from an electric resistance meter with pins embedded at 1-1/8” depth at two locations on the specimen face and one location on the specimen edge, within the interior layer. All of the panels were cut into squared specimens with variable dimensions, for a total of 168 specimens.

Steel rods measuring 250 mm in length were cut from 1000 mm long FT zinc-coated steel rods. These included both 4.8 strength-class rods with a manufacturer-specified ultimate characteristic strength of 400 N/mm^2, as well as 8.8 strength-class rods with a manufacturer-specified ultimate characteristic strength of 800 N/mm^2. Of each strength class, 42 rod segments were cut from rods with a 20M metric thread, and 42 with 24M metric thread, for a total of 168 rod segments. The rod threads are in accordance with DIN 975.

The grout mixture used to bond the rods to the CLT specimens was a three-component epoxy grout with a manufacturer-specified 7-day compressive strength of 103 MPa, at an ambient temperature of 23°C, in accordance with testing standard ASTM C579 B. The adhesive formula was selected for its availability, high compressive strength, and low shrinkage.

Other materials used in the testing program included 20M and 24M washers (S235 grade steel, zinc-plated, conforming to DIN 9021) and nuts (class 8 steel, zinc-plated, conforming to DIN 934).

Table 1: List of materials.

Item	Manufacturer	Description
CLT	Structurlam	V2M1.1 105 V 3-ply, 105 mm thick E1M5 105 E 3-ply, 105 mm thick
Epoxy Grout	Hilti	CB-G EG three-component epoxy grout
Threaded Rods	Rothoblaas	20M / 4.8 strength-class 20M / 8.8 strength-class 24M / 4.8 strength-class 24M / 8.8 strength-class
Nuts & Washers	Rothoblaas	20M 24M

4.2 Fabrication

Specimens were cut from the 105 mm thick CLT panels specified above in sizes of 280 x 280 mm or 340 x 340 mm. Dimensions were dictated by the minimum edge and end distances of fasteners recommended in EN 383 for bolts and dowels, based on 20 mm and 24 mm rod diameters, respectively. Each specimen had a hole drilled through its centre of which 95 mm of depth has a diameter of D and the remaining 10 mm of depth has a diameter of d, as shown in Figure 2a. The diameter d is the associated rod nominal diameter, while the diameter D corresponds to either twice, three times, or four times the associated rod nominal diameter. The specimens were then laid down on wooden rails measuring 72.5 mm thick with the smaller diameter hole facing down. Rods were placed through the holes such that the bottom end made contact with the ground, ensuring that they protruded 72.5 mm from each side of the CLT specimens. The epoxy grout was mixed according to the manufacturer's instructions and then poured into the gap between the rod and the CLT, until it was approximately half full. The grout was tamped down and then the hole was filled and the grout tamped again. Wooden rings with a thickness of 10 +/- 1 mm, with a corresponding inner diameter d and outer diameter D, were then placed in the hole and set flush to the CLT surface using a rubber mallet. This served to centre the rod and further compact the grout such that it fills any small voids. The specimens were left to cure for at least two hours, according to manufacturer recommendations, before being moved to long term storage, where they were left to continue curing for at least seven days before testing. An isometric schematic view of a fabricated specimen is given in Figure 2b.

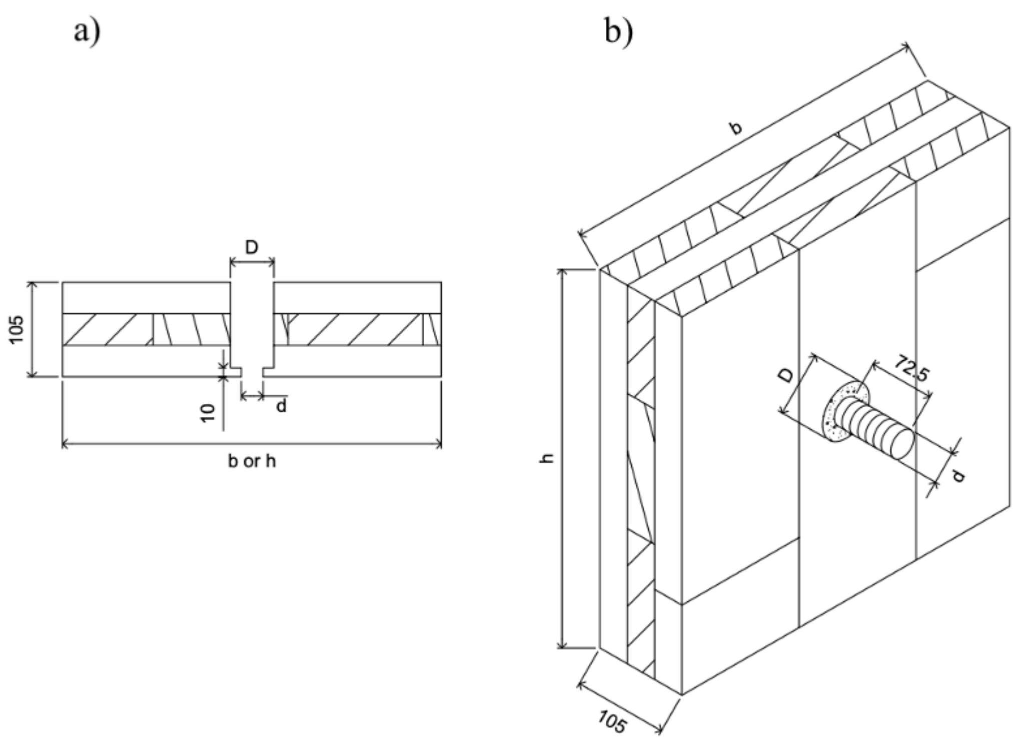

Figure 2: (a) Cross-section before rod insertion and grout fill and (b) as-built specimen in isometric view. Measurements are in mm.

Seven replicates of each possible combination of design parameters were fabricated, for a total of 168 specimens, as summarized in Table 2.

Table 2: Specimen properties.

$b \times h \times t$ (mm)	Rod d (mm)	CLT Grade	Rod Strength	Grout D (mm)	Replicates
280 x 280 x 105	20	E	4.8	40	7
				60	7
				80	7
			8.8	40	7
				60	7
				80	7
		V	4.8	40	7
				60	7
				80	7
			8.8	40	7
				60	7
				80	7
340 x 340 x 105	24	E	4.8	48	7
				72	7
				96	7
			8.8	48	7
				72	7
				96	7
		V	4.8	48	7
				72	7
				96	7
			8.8	48	7
				72	7
				96	7
TOTAL					**168**

4.3 Testing

The testing program consisted of 168 push-out shear tests in a double-shear configuration. Specimens were tested using a custom-built test jig placed in a universal testing machine (UTM) (Figure 3). The jig consists of a 105mm x 105mm x 105 mm welded steel box, two loading arms, and a platform with welded angle sections to hold the specimen in place. At their top end, the loading arms have four bolts holes (arranged two by two) by which they are connected to the box with high-strength bolts and nuts, and at their bottom end, three bolt holes aligned vertically, corresponding to the three rod diameters of 20 mm, 24 mm (phase 1), and 30 mm (phase 2). The jig is fastened to the specimen's rod with nuts and washers tightened without preloading and in such a way that it loads the rod symmetrically and imparts only shear forces. Two linear voltage displacement transducers (LVDTs) were used to measure the relative displacement between the steel rod and the CLT panel. Load was captured by the UTM load cell. Specimens were only tested in their strong direction, with a 0° load-to-grain angle with respect to the external layers. Displacement-controlled loading was applied monotonically at a rate of 1.2 mm/min or 1.8 mm/min in order to reach maximum load within 10 minutes, in accordance with testing standard ASTM D5764 (ASTM 2018). Testing was conducted until failure or a relative displacement of 15 mm was reached, whichever occurred first.

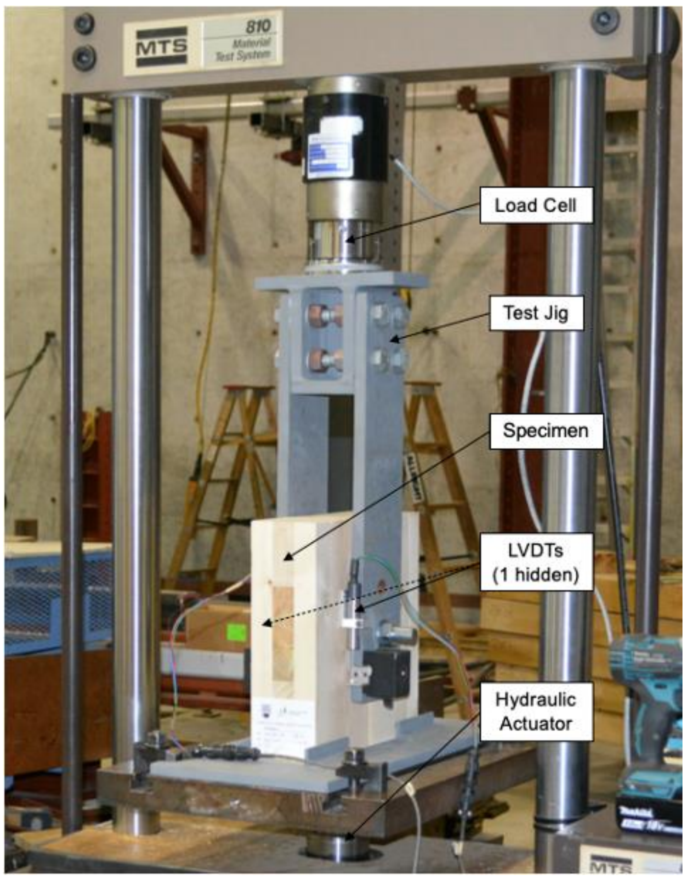

Figure 3: Test setup.

4.4 *Structural Performance Assessment*

The three structural performance parameters identified within this project were the maximum strength, ultimate strength, and stiffness of the connection. The load-carrying capacity has been taken as the maximum load recorded during testing, while the ultimate resistance has been taken as the load at actual failure, the load at 15 mm displacement, or 80% of the maximum load, whichever occurred first. The elastic stiffness of the connection, k_i, has been assessed using Equation 1, in accordance with the European standard EN 26891 (CEN 1991).

$$k_i = \frac{0.4F_{max} - 0.1F_{max}}{\delta_{04} - \delta_{01}} \quad [1]$$

Where F_{max} represents the maximum load recorded, δ_{04} represents the displacement at 40% of the maximum load, and δ_{01} represents the displacement at 10% of the maximum load.

5 Results and Discussion

5.1 Behaviour and Failure Mechanisms

All specimens exhibited linear elastic behaviour under small load amplitudes. After yielding, plastic deformation was observed as a combination of wood crushing and steel rod hinging. As expected, more hinging was observed in specimens with 4.8 strength class steel rods compared to specimens with 8.8 strength class steel rods, as well as in specimens with 20M steel rods compared to 24M steel rods (Figure 4). All of the specimens with 8.8 rods reached the target 15 mm displacement without failing, although suffering severe splitting in the CLT face (Figure 5). Additionally, specimens with 4.8 strength class 20M steel rods in E grade CLT with a grout diameter twice the rod diameter did not fail, nor did specimens with 4.8 strength class 20M steel rods in V grade CLT with a grout diameter twice or three times the rod diameter. Specimens with 4.8 strength class 20M steel rods in E grade CLT with a grout diameter three or four times the rod diameter, as well as specimens with 4.8 strength class 20M steel rods in V grade CLT with a grout diameter four times the rod diameter, failed before 15 mm displacement due to shear failure of the steel rod (Figure 6).

Figure 4: Comparison of plastic hinging in specimens with a 20M steel rod (left) and 24M steel rod (right).

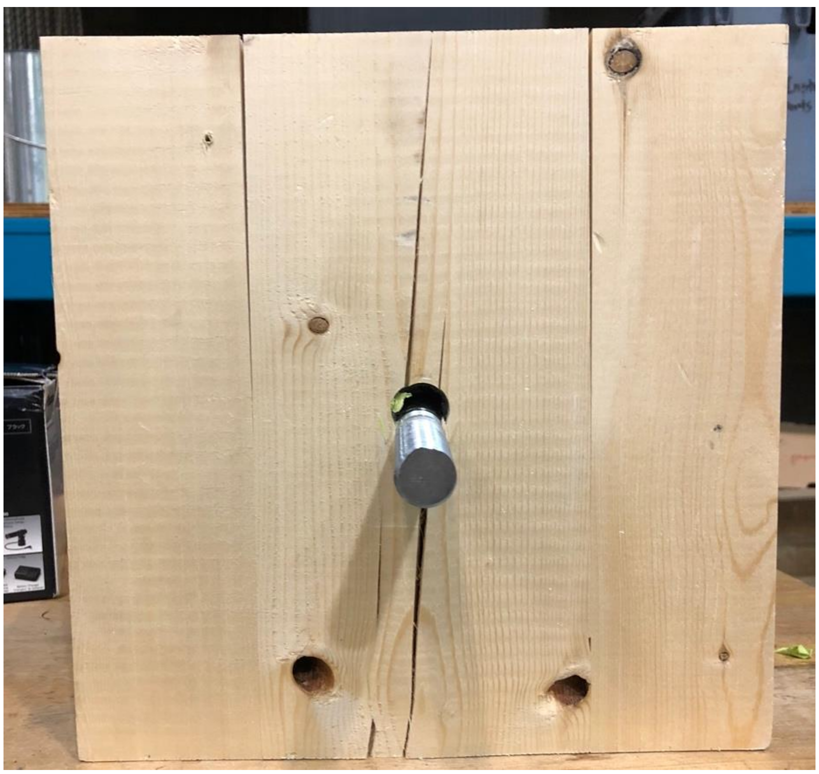

Figure 5: Splitting typical of a specimen with strength class 8.8 steel rods.

Figure 6: Shear failure in steel rod.

5.2 Load-deformation behaviour

The load-deformation curves for each combination of rod and grout diameter, obtained as an average of the seven replicates, are given in Figure 7 to Figure 12. Each chart shows the curves for the E grade (blue) and V grade (orange) CLT specimens corresponding to a steel strength class of 4.8 (solid) or 8.8 (dashed).

In general, the specimens with 20M rods exhibited a very smooth elastic-to-plastic transition, making it difficult to identify the exact point at which yielding occurred. The use of strength class 8.8 rods tended to increase the elastic range and sharpen the transition to plastic behaviour. Additionally, the specimens with strength class 4.8 steel exhibited a plateau after yielding, whereas those with strength class 8.8 steel exhibited softening. The same is true of the 24M specimens, with the specimens with strength class 8.8 steel rods exhibiting a more obvious yield point and softening afterward. The specimens with strength class 8.8 steel rods also exhibited higher ductility, with none failing by the 15 mm target displacement. In general, the grade of the CLT did not make a noticeable difference on specimens with small grout dimeters, while for specimens with large grout diameters the E grade CLT performed better. Comparing rod diameter, similar behaviour is observed, with the 24M specimens reaching a higher maximum force (stronger) and having a steeper elastic portion (stiffer).

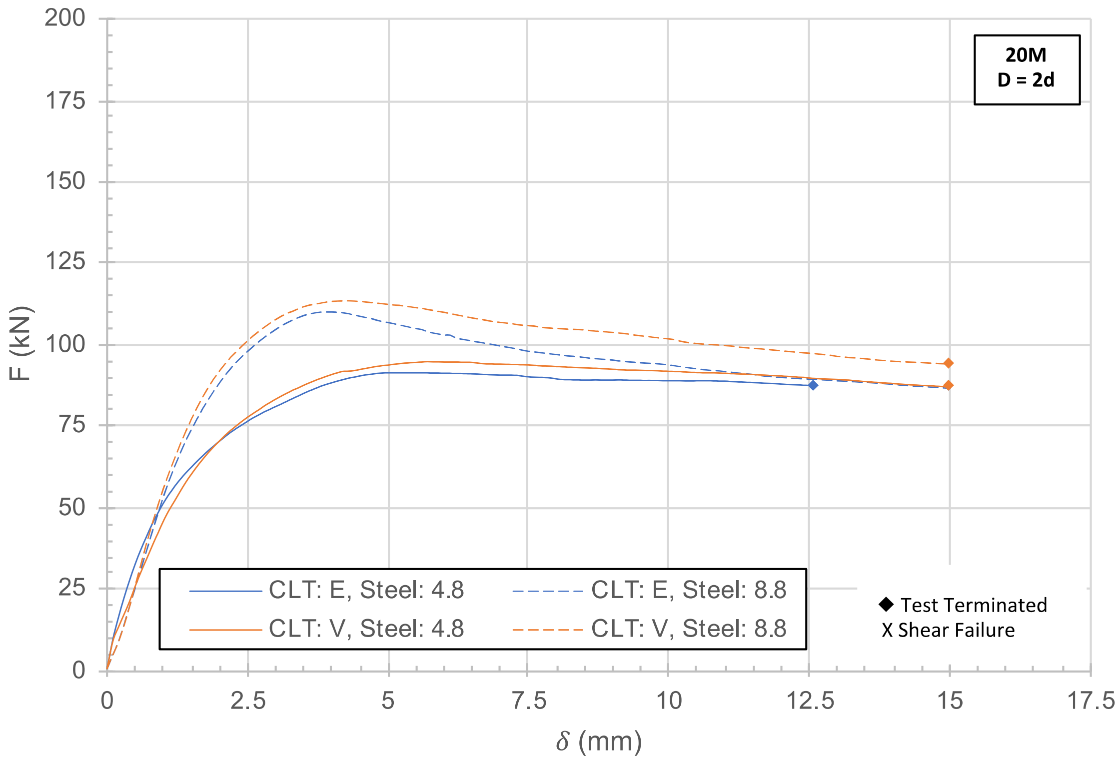

Figure 7: Load-deformation curves for specimens with 20M steel rods and D = 2d.

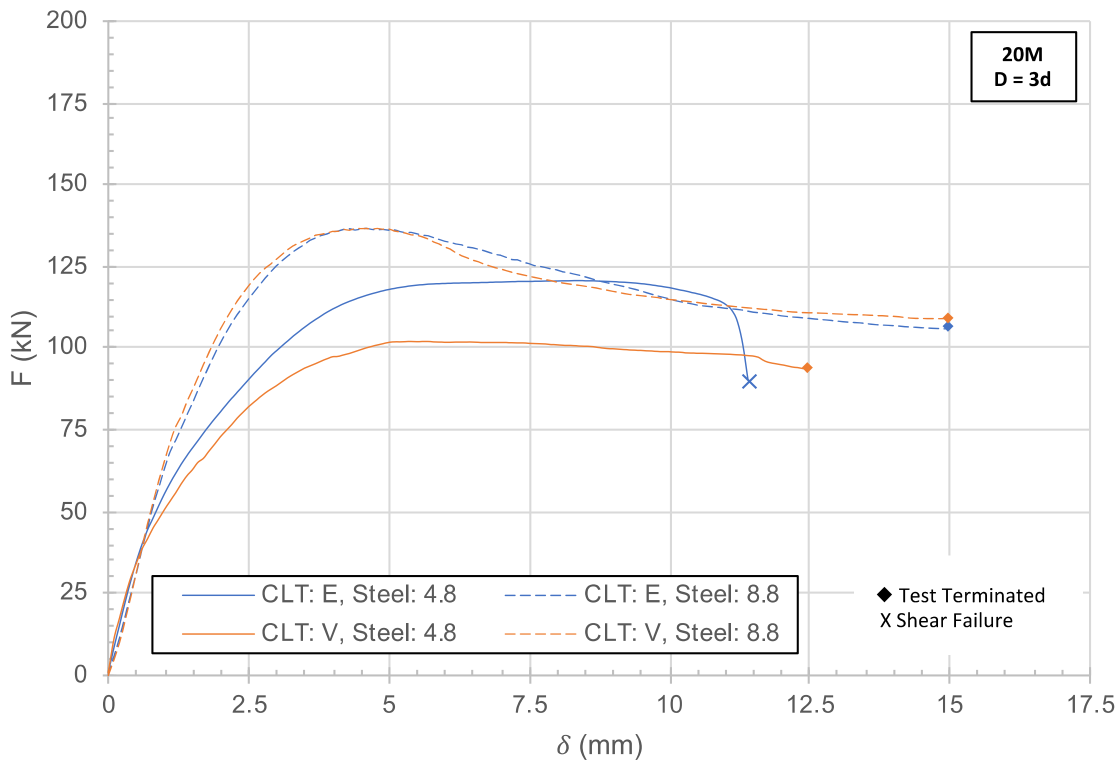

Figure 8: Load-deformation curves for specimens with 20M steel rods and D = 3d.

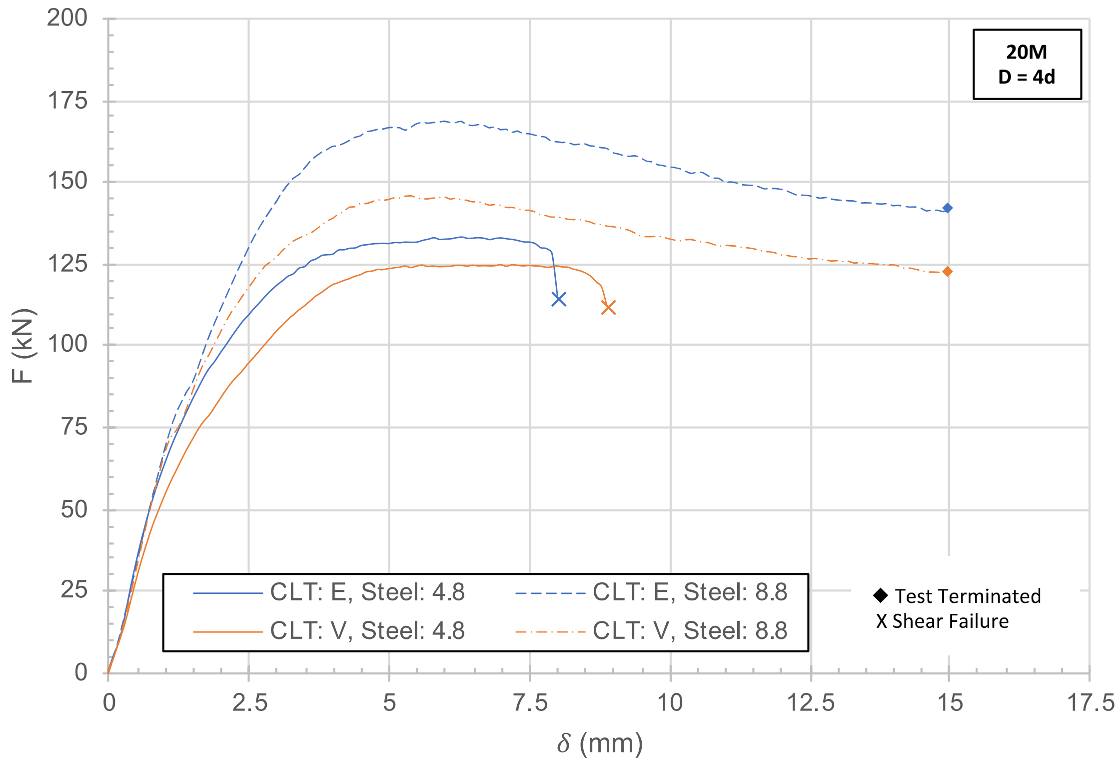

Figure 9: Load-deformation curves for specimens with 20M steel rods and D = 4d.

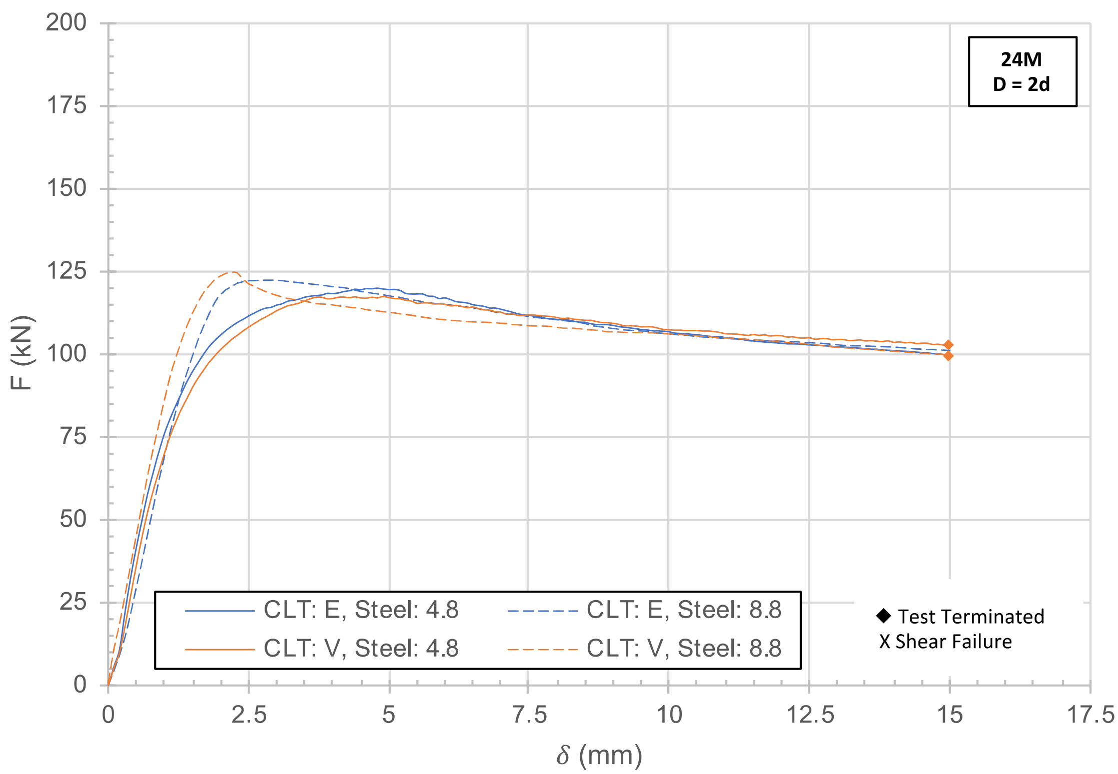

Figure 10: Load-deformation curves for specimens with 24M steel rods and D = 2d.

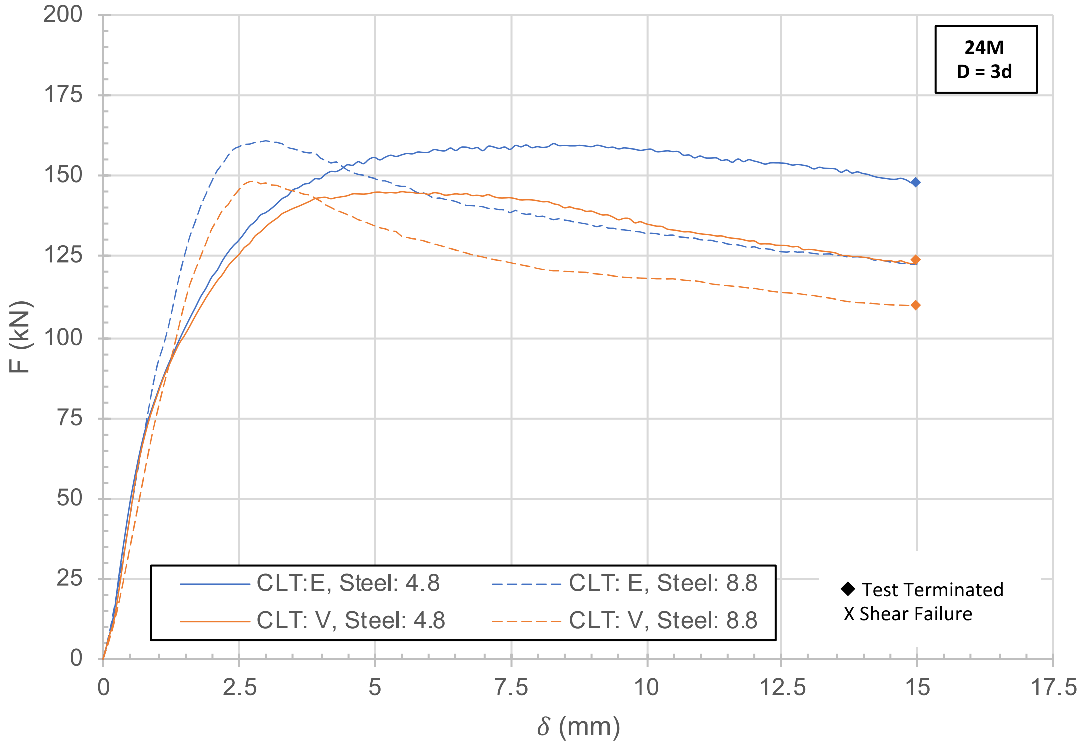

Figure 11: Load-deformation curves for specimens with 24M steel rods and D = 3d.

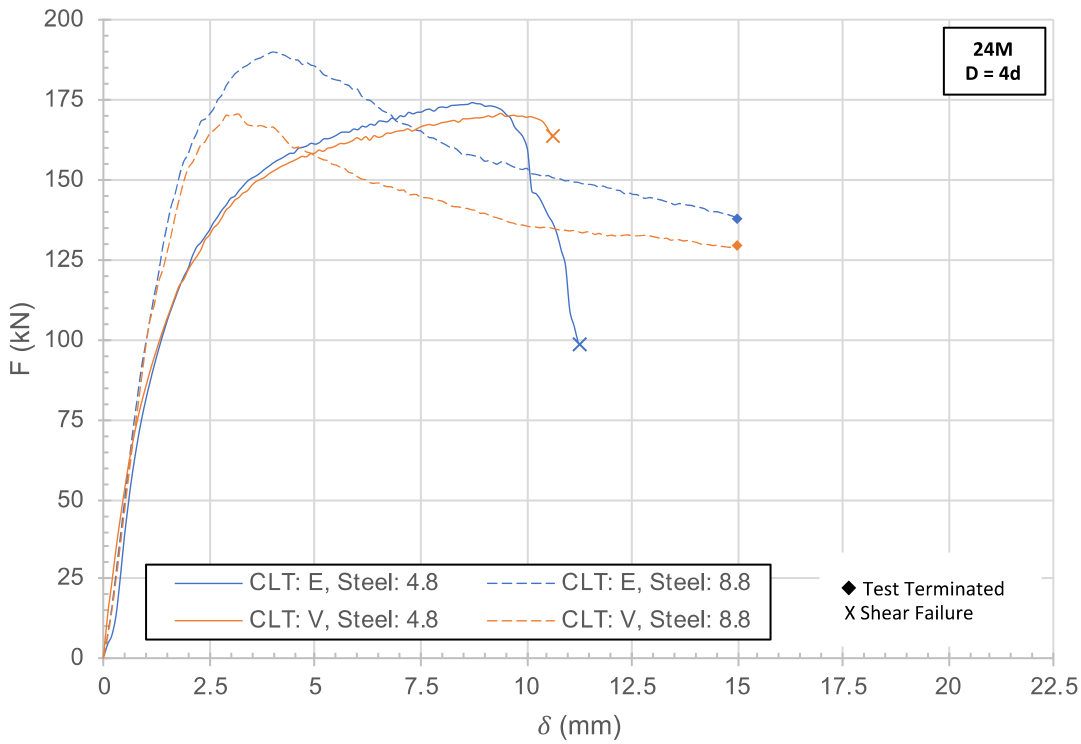

Figure 12: Load-deformation curves for specimens with 24M steel rods and D = 4d.

5.3 Structural Performance

The structural performance parameters derived from the experimental dataset are summarized in Table 3. As expected, specimens with larger diameter and stronger strength-class rods, as well as with thicker grout layers, exhibited higher strength and stiffness performance. Specifically, specimens with 24M rods are, on average, 23% stronger and 60% stiffer than specimens with 20M rods. The impact of the strength class of the steel rod on the specimen's strength varied from no increase to an increase of 33% when comparing specimens with rods in strength class 8.8 to specimens with rods in strength class 4.8. Increasing the grout diameter from twice to three times the rod diameter caused an average increase of strength of 24%, while increasing the grout diameter from three to four times the rod diameter caused an average increase of strength of 15%.

Table 3: Structural performance parameters.

Rod d (mm)	**CLT Grade**	**Rod Strength**	**Grout D (mm)**	F_{max} **(kN)**	F_u **(kN)**	k_i **(kN/m)**
20	E	4.8	40	91	87	55
			60	120	108	55
			80	133	128	73
		8.8	40	110	88	56
			60	137	110	66
			80	169	141	71
	V	4.8	40	95	87	42
			60	102	93	51
			80	125	118	57
		8.8	40	113	94	62
			60	137	110	73
			80	146	122	73
24	E	4.8	48	120	100	97
			72	160	148	103
			96	174	139	108
		8.8	48	123	102	73
			72	161	128	98
			96	189	152	111
	V	4.8	48	117	102	86
			72	145	123	110
			96	171	137	91
		8.8	48	125	100	87
			72	149	119	84
			96	170	136	106

6 Conclusions

Phase 1 of this research project studied the behaviour of 168 shear connectors consisting of FT steel rods embedded in CLT panels with a ring-layer of epoxy-based grout. The specimens' rod diameter, steel strength class, grout thickness, and CLT grade were all varied and tested in every possible combination, with seven replicates each. The specimens were subjected to a monotonic push-out shear test in order to observe their behaviour and record their load-deformation properties. It was found that the load-carrying capacity and stiffness of the connection increased with rod diameter. The resistance also increased with steel strength and grout diameter. Observed behaviour included initial elastic deformation followed by yielding and plastic behaviour characterized by a combination of steel rod hinging and wood embedment. Specimens with lower strength class steel and large grout thickness tended to fail suddenly due to shear failure of the steel rod. Specimens with higher strength class steel all exhibited desired ductility, reaching the 15 mm target displacement without failing.

The fabrication of the specimens occurred completely in the lab with common machining tools and specimens were fastened to the test jig by using nuts and washers as per common bolted connections, making them easily removable. This indicates that this type of connection is conducive to off-site prefabrication and on-site disassembly.

The connection has multiple variable design parameters, making it intrinsically versatile, allowing for its use in a variety of loading scenarios, including in forming high-performance connections in multi-storey timber-based hybrid buildings. Additionally, the specimens exhibited predictable behaviour, and in the case of the specimens with strength class 8.8 steel rods, an identifiable yield point, allowing for capacity design and making possible disassembly and reuse. Finally, the observed behaviour of the connection supports the development of reliable design models as per common bolted connections in timber.

7 Future Work

Phase 2 of this research program, which includes testing of new specimens and developing mechanics-based models and empirical design formulas, is underway and estimated to be complete by the end of the year. Future work will also include testing connections with a concrete-type adhesive (CTA) layer, as well as varying testing loading protocols.

8 Deliverables

Work done and data collected within this project has been used in preparing the conference paper "Performance of a Grout-Reinforced Hybrid Steel-Timber Shear Connection for Mass Timber Buildings" to be presented at the 2021 CSCE (Virtual) Annual Conference, as well as a poster that was awarded 1st place in the 2020/21 Forestry Grad Student Research Poster Competition. The work also received endorsement from the Canadian Wood Council (CWC) through the 2021 Catherine Lalonde Memorial Scholarships Award. This work will also contribute to the primary author's master's thesis, which will be available online at the UBC thesis and dissertation repository by the end of 2021 (https://open.library.ubc.ca/cIRcle/collections/ubctheses).

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10 Annex A

10.1 Materials

Figure A-1: CLT panels before to be processed.

Figure A-2: Steel rods cut into lengths of 250 mm.

Figure A-3: Rings for centering rods laser-cut from plywood panel.

Figure A- 4: Pair of buckets of the epoxy-based grout used.

10.2 Processing and Fabrication

Figure A-5: CLT panel being processed in the Hundegger ROBOT-Drive CNC machine.

Figure A- 6: Processed CLT panels ready for fabrication.

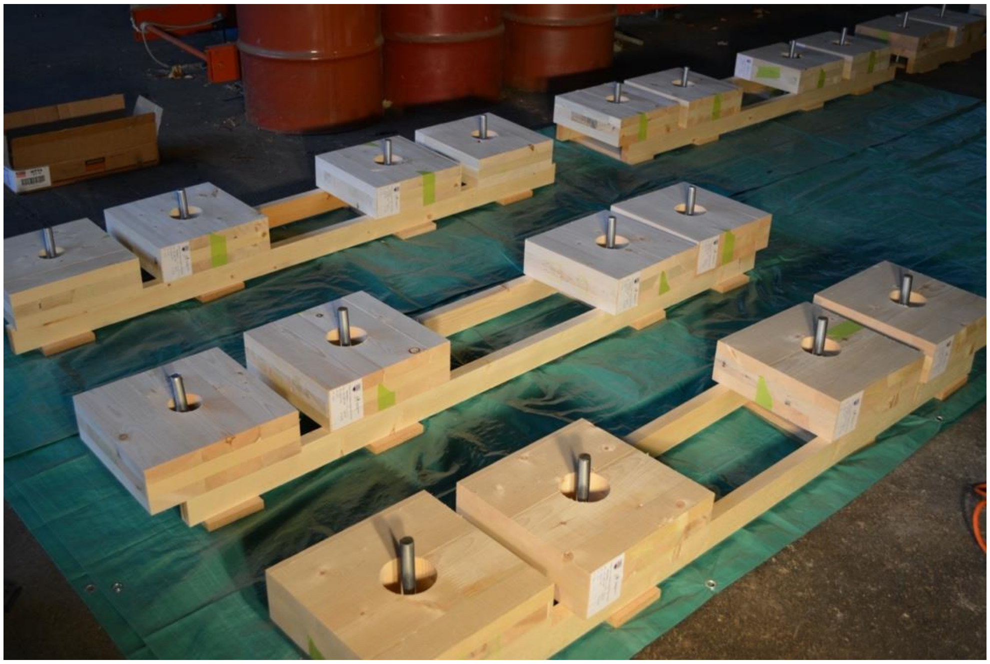

Figure A-7: Specimens ready to be filled with grout.

Figure A-8: Specimens filled with grout and sealed with wooden ring.