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Advancing Timber for the Future Built Environment

A NOVEL BUCKLING-RESTRAINED CONNECTION FOR MASS TIMBER BRACED FRAMES AND CLT SHEAR WALL HOLD DOWNS

Dylan C. Neves[1], Joshua Woods[2]

ABSTRACT: Mass timber structures rely on the connections for stiffness, strength, ductility, and energy dissipation capacity in the event of an earthquake. Common connection types for timber braced frames and hold-downs in cross-laminated timber (CLT) shear wall buildings include dowel-type and self-tapping screw connections that can exhibit ductile behavior but have a highly pinched hysteretic response and lead to significant damage to the timber element. This paper presents a novel buckling restrained connection for mass timber structures that attempts to mitigate some of these challenges. The connection combines a glued-in rod with a steel buckling restrained axial fuse to form a high stiffness and strength connection that is fully concealed within the timber. This paper discusses proof-of-concept testing of the connection and examination of the influence of the axial fuse core length and diameter on the connection response. The experimental results indicate that yielding is concentrated in the fuse core resulting in stable hysteretic behavior without pinching, leading to larger energy dissipation capacity when compared with traditional timber connections. System-level seismic assessment of the connection is also evaluated for a CLT shear wall building using nonlinear time-history analysis. The results demonstrate significant potential for the proposed connection to improve seismic performance.

KEYWORDS: Mass timber, braced frames, CLT shear wall, connections, axial fuse, glued-in rod, time-history analysis

1 – INTRODUCTION

Over the last decade, there has been a rapid increase in the number of mass timber buildings constructed in North America [1], largely due to the environmental benefits of building with wood. Despite significant advances in the manufacturing, analysis, and design of timber structures, questions remain surrounding the performance of tall wood structures under extreme load events, including earthquakes. Much of the recent research has focused on traditional dowel-type or self-tapping screw connections, however, there is a continued need to develop new high-stiffness, strength, and ductility connections for tall mass timber structures.

The goal of this research is to design, construct, and test a novel connection for mass timber structures. This connection combines a typical glued-in-rod with a steel buckling restrained axial fuse to form a buckling-restrained connection. The target behavior of the connection under cyclic loading is to mimic the performance of a buckling restrained brace, a common lateral force resisting system used in steel-only construction. The long-term goal is to integrate this connection into timber structures to form a highly ductile seismic force-resisting system, with a ductility related modification factor that is comparable to those that might be used in steel-only construction. The specific objectives of this paper are to: (1) assess the feasibility of combining a glued-in-rod and buckling restrained axial fuse to form a high- stiffness, strength, and ductility connection; (2) construct and test a series of prototype connections with a range of capacities, (3) develop a nonlinear spring that is capable of simulating the behaviour of the connection, and (4) conduct nonlinear time-history analysis on a full-scale prototype structure utilizing the connection and compare its behaviour to a more traditional self-tapping screw hold-down connection.

2 – BACKGROUND

Research related to highly ductile and resilient steel structures has been ongoing for several decades. For example, researchers have focused on the application of energy dissipative braces in steel braced frames [2-3], yielding links in moment-resisting and eccentrically braced frames [4-6], and rocking systems with dissipative elements [7-8]. Alternatively, reinforced concrete structures have traditionally relied upon the formation of plastic hinges (i.e., yielding of steel reinforcement coupled with concrete cracking and crushing) to dissipate seismic energy and provide system ductility during an earthquake.

[1] Dylan Neves, Graduate Student, Department of Civil Engineering, Queen's University, Kingston, Canada, 18dcn@queensu.ca

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

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In mass timber structures, the most used lateral load resisting systems include cross-laminated timber (CLT) shear walls and braced or moment-resisting frames. Because wood can act in a brittle manner when loaded in the perpendicular-to-grain direction, the yielding elements in conventional CLT shear wall and frame type structures have been the connections, which often rely on steel fasteners to yield and dissipate energy. In CLT shear wall systems these connections include the hold-downs at the toes of the CLT panel and the panel-to-panel connections. These connections have traditionally been nailed [9], but researchers have also examined bolts [10] and self-tapping screws [11]. In braced frames, the yielding elements are the brace connections, which have used rivets [12], dowels [13-15], or bolts [16-17]. While these connections types can exhibit a ductile yielding failure mode if adequately detailed, challenges with these connection types include a highly pinched hysteretic response that limits energy dissipation, but also very conservative design methods in existing design standards, which leads to concerns surrounding the ability of engineers to accurately predict their strength, something that is critical to ensure other elements in the structure can be capacity protected and collapse can be prevented.

3 – EXPERIMENTAL PROGRAM

The goal of this study was to conduct proof-of-concept testing on the proposed connection. First, tests were conducted on axial fuses, to verify their performance prior to moving to connections that combined the axial fuse with a glued-in rod embedded into timber. Table 1 summarizes the design details for the connections tested in this study. Specimen names beginning with an 'F' in Table 1 are standalone axial fuses, and the number in the name is the target design strengths, which were 25 kN and 50 kN. Both fuses had a core length of 152 mm, which is relatively short when considering the axial fuses that have been commonly studied in the literature. By using shorter core lengths this reduces the required depth of the hole, potentially improving constructability of the connection. However, this shorter core length could compromise the connection elongation capacity, and so, this is something of interest in this study.

The axial fuse tests also considered the influence of loading protocol on their behaviour, recognizing that depending on whether the connection is used in a brace connection or in a CLT shear wall hold-down, this would change the loading conditions. Consequently, one axial fuse was tested under a tension-only loading protocol (R = 0), referred to as specimen F-50-R0. This loading protocol is more representative of the load the connection would experience in a CLT wall hold-down, in which the connection would only transfer load in tension.

Table 1: Test matrix and connection details

Specimen Name	d_{rod} *(mm)*	L_e (mm)	d_{core} (mm)	l_{core} (mm)	P_{design} (kN)
F-25	12	-	8	152	25
F-50	16	-	12	152	50
F-50-R0	16	-	12	152	50
W-25	12	100	8	152	25
W-50	16	150	12	152	50

Note: all of the specimens were ASTM A305 steel rod

Fig. 1 shows a typical axial fuse used in this study. The axial fuses were all fabricated of steel threaded rod, which had a reduced section, referred to as the core, as well as a threaded section that extended on either end of the axial fuse, which were used to facilitate the connection to the test machine (or used as glued-in rod).

Specimen names beginning with a 'W' in Table 1 are connections that included a glued-in rod and axial fuse embedded into a glued-laminated (glulam) timber member. Fig. 2 shows the geometry of the tested connections. The glulam was 24f-E SPF. This study investigated 2 different core diameters, which were associated with the 2 design strengths of 25 kN and 50 kN, as noted in the specimen names. The glued-in rod length for each specimen, which are shown in Table 1, were determined using the approach suggested by Stieger et al. [18], which is shown in Eq. (1):

$$P_{r,rod} = 7.8 \left(\frac{\lambda}{10}\right)^{-1/3} \left(\frac{\rho}{480}\right)^{0.6} \pi d_h \, L_e \qquad (1)$$

where λ is the ratio of hole diameter to hole depth, referred to as the slenderness ratio, ρ is the wood density (kg/mm^3), d_h is the hole diameter, and L_e is the embedment length. In the practical design of this connection type, the glued-in rod would be capacity protected to prevent brittle failure and ensure yielding of the fuse core. This was accomplished in these proof-of-concept tests by designing the glued-in rod connection for a load that was 1.3 times the design load in Table 1.

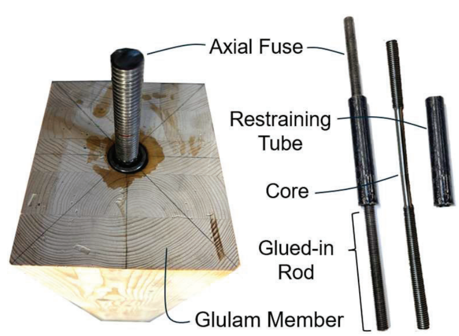

Figure 1: Construction of a typical specimen

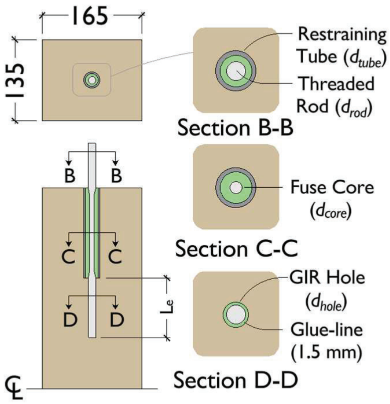

Figure 2: Geometry of tested connections

Construction of the connections included first drilling a stepped hole in the timber with two different diameters, including one hole sized for the glued-in rod and a second hole for the restraining tube of the axial fuse. After drilling, an epoxy resin was injected into the hole and the axial fuse was inserted into the timber member and left to cure for at least a week prior to testing. A typical completed specimen is shown in Fig. 1.

3.1 EXPERIMENTAL SETUP

Fig. 3 shows the experimental test setup. The specimens were tested in a universal testing frame (+/- 500 kN, +/- 300 mm). The specimens were secured using the hydraulic grips and subject to cyclic loading. Table 2 summarizes the cyclic amplitudes in the loading protocol. The protocol is based on the qualification testing protocol for steel buckling restrained braces, which is described in detail by [19]. The protocol includes applying decreasing numbers of cycles at increasing increments of the yield displacement (Δ_y).

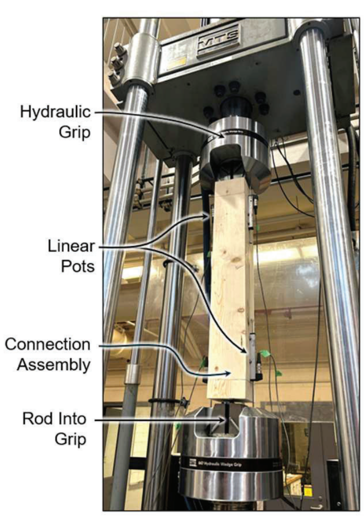

Figure 3: Annotated photo of experimental test setup

Table 2: Loading protocol (adapted from [19])

No. Cycles	**8**	**6**	**4**	**3**	**2**	**2**
Amplitude	Δ_y	$1.5\Delta_y$	$2.5\Delta_y$	$3.5\Delta_y$	$4.5\Delta_y$	$5.5\Delta_y$

To measure the structural response of the connections, they were instrumented with 2 linear potentiometers on each end of the member to measure the local displacement of each connection. The applied load was measured using a load cell connected to the hydraulic actuator. Data was sampled at 5 Hz for all tests.

3.2 EXPERIMENTAL RESULTS

Table 3 summarizes the results from the tested specimens, including the yield load and displacement, ultimate load and displacement, and the displacement ductility, which was determined as the ratio of the ultimate displacement and the yield displacement.

Fig. 4 shows the force versus displacement hysteretic response of the axial fuse specimens. The results show that all fuses exhibited a stable hysteretic response with yielding in both tension and compression. The yield loads of the specimens of 22.8, 51.0, and 49.0 kN are within 5% of the target design load on average, demonstrating the accuracy with which the strength of these fuses can be determined, something that is difficult to achieve in conventional timber connections (e.g., bolted or screwed connections). The axial fuses achieve ultimate displacements of 3.2 and 8.0 mm, which represents between 2 % and 5 % of the core length. The results demonstrate that a core length of 150 mm is likely not sufficient to achieve the required connection displacement demand, however, longer core lengths could be used in practice, and the goal of this work was a proof of concept of the connection type. The specimen tested under tension-only cyclic loading had a larger maximum displacement of 12.7 mm, representing 8% elongation. Typical hold down connections might be expected to have displacement demands between 25 and 40 mm, and so, core lengths between 300 and 500 mm would likely be required for steel axial fuses in practice.

Table 3: Summary of experimental test results

Specimen Name	P_y **(kN)**	Δ_y **(mm)**	P_u **(mm)**	Δ_u **(mm)**	**µ**
F-25	22.8	0.4	27.0	3.2	8.0
F-50	51.0	0.6	63.0	8.0	13.2
F-50-R0	49.0	0.5	60.0	12.7	25.4
W-25	22.5	0.9	28.0	4.5	5.0
W-50	53.5	1.2	78.5	12.8	10.6

Note: values are average of tension and compression

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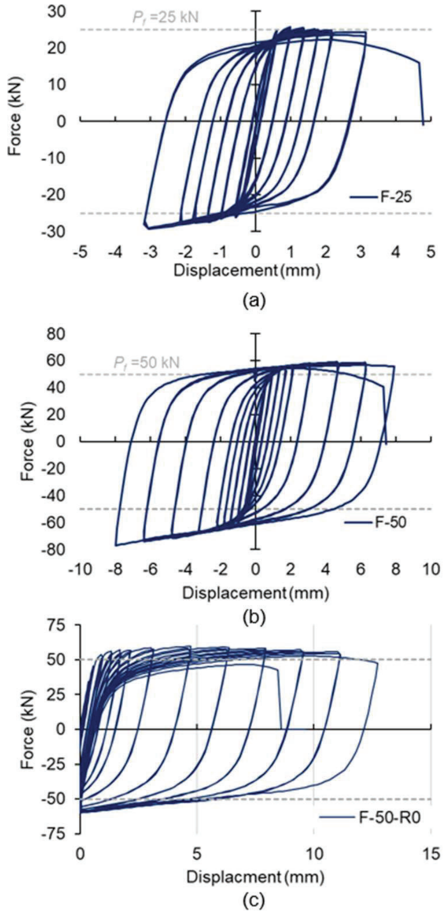

Figure 4: Hysteretic response: (a) F-25, (b) F-50, (c) F-50-R0

Fig. 5 shows the force-displacement hysteretic response of the connection assemblies tested that included the glued-in rod, axial fuse, and timber element. Table 3 includes the key structural response parameters for these connections as well. The results show that once again both connections achieve yield loads that were within 10% of the design load and had very stable hysteresis loops without pinching. The strength of the connections is higher on the compression side, which is expected because of friction and Poisson's effect.

The results in Table 3 are for the full member response, and so, the displacements include the displacement of both connections. The results show that the ultimate displacement of the connections is 40-60% higher than the individual axial fuse tests, indicating that two connections in series will not produce twice the displacement capacity. In this case, examining the individual connection responses, the results showed that both connections exhibited maximum individual displacements of approximately 6 mm, indicating that both connections were engaged during loading.

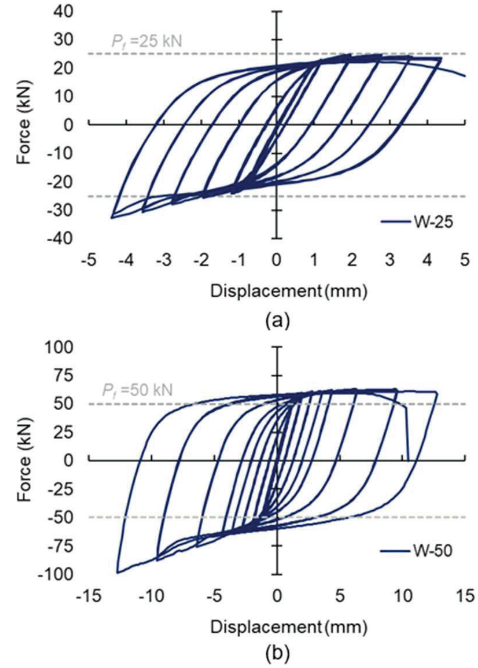

Figure 5: Hysteretic response: (a) W-25 and (b) W-50

Failure of the connection specimens occurred as a result of fracture of the axial fuse core in tension, as shown in Fig. 6. In both tests no brittle fracture was observed in the glued-in rod connection, indicating that the adopted capacity design approach was effective. Ultimately, these proof of concept tests suggest that while a longer core length is required to achieve the displacement capacity required for connections in mass timber structures, the connection concept is effective and has several characteristics that are beneficial when compared with traditional timber connections, including high stiffness and strength, stable hysteretic loops without pinching, and the ability to accurately predict its strength, especially if the yield strength of the steel is known.

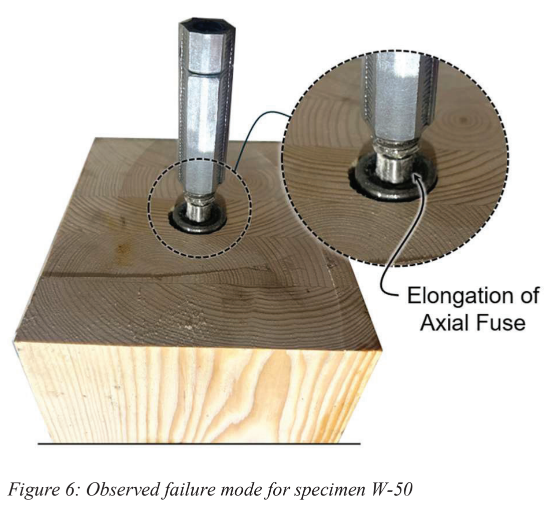

Figure 6: Observed failure mode for specimen W-50

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4 – NUMERICAL MODELING

4.1 – NONLINEAR CONNECTION MODEL

Based on the proof-of-concept testing of the buckling restrained connection, the results show that the connection can achieve the target design load and exhibits stable hysteresis through yielding in both tension and compression. The third and fourth objectives of this paper were to: (1) develop a nonlinear spring that could replicate the behaviour of the buckling restrained connection and (2) compare the response of a full-scale buckling restrained hold-down connection to a more traditional self-tapping screw hold-down connection through nonlinear time-history analysis.

To accomplish these objectives, the OpenSees platform was utilized, which is an open-source finite element software that has been used extensively in the literature to simulate the response of structural systems subjected to earthquakes. Models of two connection types were developed using a simple model of a beam-column element connected to a zero-length element. The zero-length element was assigned a nonlinear uniaxial material model that depended on the connection type. One end of the zero-length element was fixed while the other was connected to the beam-column element. A tension-only reversed cyclic loading protocol was applied at the beam tip (to simulate hold-down loading).

To model the behaviour of the buckling restrained connection, the *Steel02* material model was utilized and compared to experimental data from test "F-50-R0". *Steel02* is a Giuffré-Menegotto-Pinto Model with Isotropic Strain Hardening [20]. Definition of the model includes seven parameters, three of which control the transition from elastic to plastic behaviour, while the other four control isotropic hardening. Fig. 7 compares the hysteretic behaviour from the experiment to the nonlinear material model. The results show that the model does an adequate job at capturing the strength and stiffness of the connection during loading and unloading.

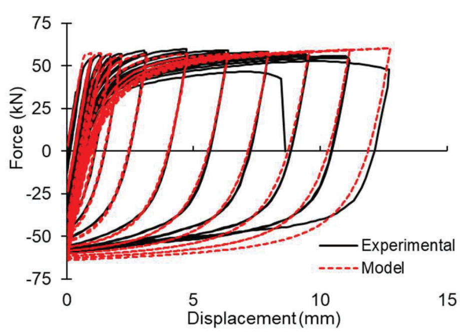

Figure 7: Buckling restrained connection modelling results

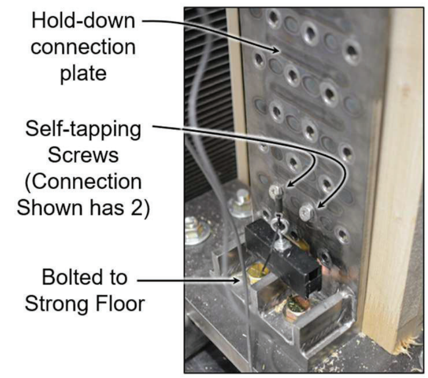

Figure 8: Self-tapping screw connection modelling results (image from Pan et al. [21])

In addition to developing a nonlinear model of the buckling restrained connection, a model of a traditional self-tapping screw hold-down connection was also developed in order to enable comparison of the system-level response of a CLT shear wall using time-history analysis. Data from a study by Pan et al. [21] was used for the calibration, which is shown in Fig. 8. The tested hold-down connection included a thick steel bracket that was fastened to a 5-ply CLT panel (139 mm thick) with 6 – 12 mm diameter and 120 mm long self-tapping screws (ASSY Kombi LT). The specimen was tested in direct tension under a tension-only reversed cyclic loading. For more information on the testing, see Pan et al. [21].

The nonlinear response of the self-tapping screw hold-down connection was modelled in OpenSees using the *Pinching4* uniaxial material model, which uses an input backbone curve and 16 unique parameters to control stiffness and strength degradation under cyclic loading as well as the pinching in the response. Fig. 9 compares the experimental results to the numerical model. The results show that the model is able to capture the nonlinear behaviour of the connection with reasonable accuracy, including the backbone curve under cyclic loading, stiffness degradation, and pinching.

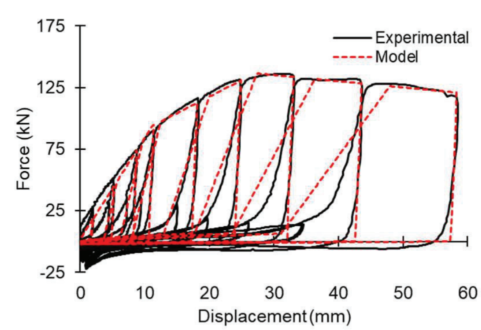

Figure 9: Self-tapping screw connection modelling results (experimental results from Pan et al. [20])

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In addition to the buckling restrained connection, a self-tapping screw connection was also sized. Based on the results from [21], the connection had 8 – 12x120 mm self-tapping screws, which resulted in a connection capacity of 165 kN. It is noted that if the capacity of the connection was determined according to CSA O86, the connection capacity would likely be estimated to be much smaller as results demonstrated by [21] suggested that the strength of these connection types is as much as 3 times higher than the predicted strength. Nonetheless, to make an apples-to-apples comparison, the strength was based on the experimental results.

4.2 – ARCHETYPE BUILDING DESIGN

To evaluate and compare the system-level performance of each connection type, an archetype building was designed according to the 2020 National Building Code of Canada (NBCC) and CSA O86-19 [22,23]. Fig. 10 shows a plan view of the structure. The building was a balloon type CLT shear wall structure located in Vancouver, British Columbia, Canada. The floor area was 600 m^2, which included 4 bays in each direction. Each of the six storeys was 3 m tall, resulting in an overall building height of 18 m. The CLT shear walls were 6 m wide and 18 m tall. The dead load was 2.5 kPa for a typical floor and 1.4 kPa on the roof. The snow load was 3.0 kPa. The seismic weight of the structure was taken as the dead load plus 25% of the snow load (1.0D+0.25S).

The number of required shear walls was determined using the Equivalent Static Force Procedure (ESFP) in NBCC. Given that balloon type CLT shear walls are currently not recognized in the NBCC as a seismic force resisting system, the ductility or overstrength modification factors (R_d and R_o) were assumed to be 4.0 and 1.5, respectively. Based on the results, ten shear walls were required in each direction to carry the seismic load. The CLT panels were 9-ply (315 mm thick), which was governed by the in-plane shear capacity of the panel.

Based on the seismic loads and the overturning moment at the CLT shear wall base, the required hold-down force was 145 kN. To meet this demand, two different connections were designed. First, a buckling restrained connection was detailed, which used four axial fuses fabricated from 16 mm diameter rods. Assuming a yield stress of 400 MPa, this resulted in a required core diameter of 11 mm and core area of 95 mm^2 (38 kN per fuse and 152 kN overall). The core length was selected as 250 mm, to provide 25 mm of elongation capacity.

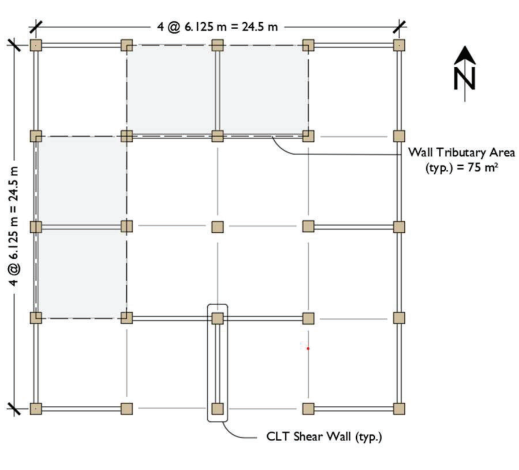

Figure 10: Plan view drawing of archetype building

4.3 – CLT SHEAR WALL MODEL

To simplify modelling of the archetype building and assuming rigid diaphragm behaviour, a single shear wall was modelled in two-dimensions, assuming that the walls do not contribute stiffness or strength in the out-of-plane direction. The CLT shear wall was modelled using plane stress elements (*quad* element). The hold-down connections were modelled using zero-length elements at the ends of the wall, which were assigned the same parameters as those discussed in Section 4.1. Finally, to model the hard contact between the base of the CLT shear wall and the foundation, a series of compression-only springs with a stiffness that was 100 times larger than the stiffness of the hold-down connections was used. The mass corresponding to 1/10 of the total seismic weight (assuming all walls had the same stiffness) was lumped at each storey level. The gravity load associated with the tributary area of the wall was also included in the model.

4.4 –EARTHQUAKE GROUND MOTION

The prototype buildings were subjected to an earthquake ground motion record from the 1989 Loma Prieta earthquake (M_w = 6.9). The record was scaled to match the uniform hazard spectrum for Vancouver, shown in Fig. 11(b), assuming a period range from 0.3-1.0s. The scaled record had a PGA of 0.24g and a length of 20 s.

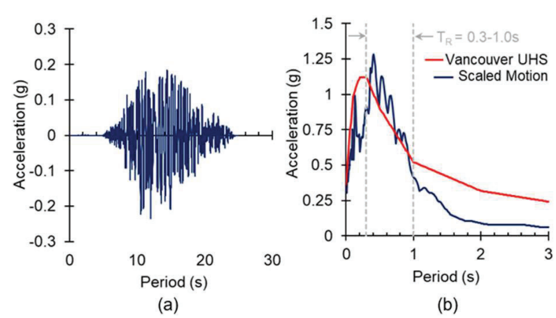

Figure 11: Scaled earthquake record: (a) acceleration-time history, (b) response spectra

4.5 – MODELLING RESULTS

Fig. 12 compares the nonlinear time-history analysis results for the two different hold-down connection types, including the top displacement of the shear wall as well as the left hold-down force versus displacement hysteretic response. The results in Fig. 12(a) show that the top displacement in the CLT shear wall with the buckling-restrained connection is 25% lower when compared to the wall with the self-tapping screw connection (74 mm versus 103 mm) and that overall, the displacements of the wall are consistently lower. This is due in large part because of the high stiffness of the buckling restrained connection, which is able to control lateral drifts during the earthquake. One observed disadvantage of the buckling restrained connection is the permanent residual drift following the earthquake, which was approximately 3 mm. This is a known disadvantage of lateral systems relying on steel yielding (e.g., buckling restrained brace frames), as the connection has a low post-yield stiffness, which leads to comparatively larger permanent drifts. It is noted that the maximum inter-storey drift for both CLT shear wall structures was less than 2.5%, which is the maximum drift limit in the NBCC [22].

Comparing the force versus displacement hysteresis for the two connection types in Fig. 12(b) and Fig. 12(c) shows that, as expected, the displacement demand on the buckling retrained connection hold-down is smaller than the self-tapping screw connection hold-down. The peak demand for the buckling restrained connection hold-down was approximately 21 mm, which is less than the design displacement demand based on the selected core length of 25 mm (250 mm core length and 10% maximum elongation). These results suggested that buckling restrained connections with a modest core length can achieve the displacement demands in full-scale structures. However, additional testing would be required to validate this performance in full-scale applications, including studying connections with multiple axial fuses combined together.

One other notable difference in the performance of the two connections would be the damage to the two connection types following the earthquake. In the case of the buckling restrained connection, all of the damage is concentrated in the axial fuse cores, while the CLT panel and glued-in rod are intended to remained elastic – a design objective that was proven to be possible through preliminary proof-of-concept testing in this study. Alternatively, the self-tapping screw connection would experience significant screw yielding, timber splitting, and bearing failure around the screws at this level of displacement, something that would be difficult to repair.

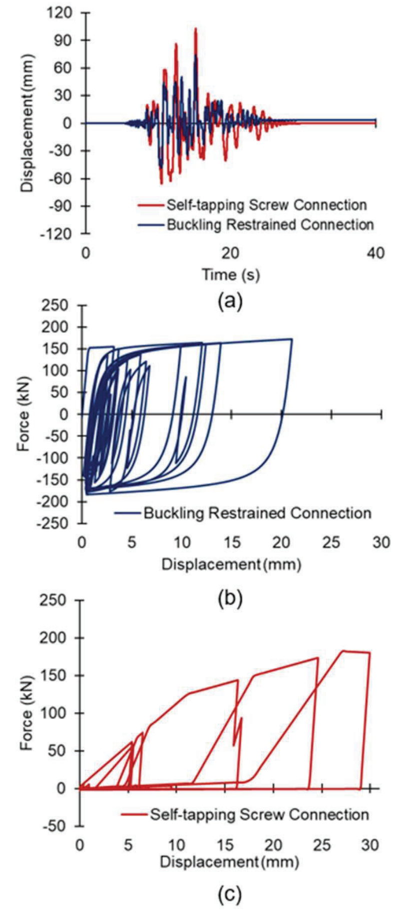

Figure 12: Comparison of seismic response: (a) shear wall top displacement, (b) buckling restrained connection force versus displacement hysteresis, (c) self-tapping screw connection force versus displacement hysteresis

5 – CONCLUSIONS

This paper presented experimental testing and numerical modelling of a new buckling restrained connection for mass timber structures. The connection combines a steel axial fuse and a glued-in rod to form a high-stiffness and high-strength connection. The advantages of this connection include the fact that it can be fully concealed within the timber, has a very predictable (low overstrength) yield load in both tension and compression when compared with traditional timber connections, and exhibits stable hysteretic behaviour without pinching. Proof-of-concept experimental testing was conducted to evaluate the cyclic behaviour of the buckling restrained connection. Furthermore, nonlinear time-history analysis was used to compare the performance of the buckling restrained connection to a self-tapping screw connection when used as a hold-down in a balloon-type CLT shear wall structure. The specific conclusions of this research are:

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1. It is feasible to combine a glued-in rod and an axial fuse to form a buckling restrained connection. Results demonstrated that the connection had high stiffness and was able to achieve a strength that was within 10% of the designs strength on average.
2. Elongation capacity of the connection was limited by the studied core lengths (150 mm), however, there is the potential to use longer core lengths in future research. Elongation capacity for steel fuses under cyclic loading was found to be roughly 10% of the core length.
3. The use of different diameter rods and different core sizes was found to be an effective approach at achieving a range of capacities. In all cases brittle failure of the glued-in rod was prevent by using capacity design principles.
4. A nonlinear model developed in OpenSees using the *Steel02* material model was found to be able to accurately simulate the cyclic behaviour of the axial fuse, permitting detailed modelling of this connection type.
5. Results from nonlinear time-history analysis indicated that the CLT shear wall structure with the buckling restrained connection had a smaller maximum lateral displacement by 25%.

Results in this study are a first step towards development of this connection type. Future work needs to consider connections with multiple axial fuses to study group effects, examination of the performance of the buckling restrained connection in CLT, and nonlinear time-history analysis using a suite of ground motion records to better understand its behaviour under earthquake loads.

6 – ACKNOWLEDGEMENTS

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