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CYCLIC BEHAVIOR OF FOUNDATION – WALL CONNECTION IN CROSS LAMINATED TIMBER SHEAR WALLS

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Abstract

Within the scope of sustainable construction, cross laminated timber (CLT) has a number of attributes that makes it an attractive alternative to build mid-rise buildings. Nevertheless, since the CLT elements are almost rigid panels, a critical matter of the seismic behavior of this construction system is the connections hysteretic response, because these links are responsible of giving a suitable ductility capacity and energy dissipation to the CLT structure subjected to seismic movement. Besides, the CLT panels in Chile are made of radiata pine timber, which is softer and lighter than the timber used worldwide to manufacture this structural material. This work presents the study of a foundation – wall connection by means of laboratory testing and numerical simulations. A set of hold down connectors and CLT walls are tested subjected to both quasi-static monotonic and cyclic loading protocols. Different fasteners configuration are analyzed, as well as load directions parallel and perpendicular to timber grain. Based on these laboratory results, two hysteretic constitutive laws (Saws models and pinching4 model) are calibrated using OpenSees simulations, taking into account the strength and stiffness degradation of the connectors, and the pinching behavior of the load – displacement curves. Results show good fitting between the simulated and the measured force - displacement curves for every connection tested and for both constitutive laws, but pinching4 model exhibits better adjustment than Saws model, particularly for the cyclic tests. Finally, using the calibrated numerical models of the hold down connectors, the cyclic behavior of a full scale 2.95 x 2.95 m CLT shear wall is explored. The simulation and test results show that the CLT panel rotates almost as a rigid body, and the nonlinear response of the connectors provides the ductility and energy dissipation for the wall under the cyclic displacement test, giving a displacement ductility over 3 with a drift capacity of almost 1,0%. Moreover, numerical model results suggest that the friction between wall and foundation should not be discarded or over simplified during structural analysis or design of CLT structures, because it provides an important part of the lateral load capacity of the wall.

Keywords: CLT shear wall; hold down connector; OpenSees

1. Introduction

Structures built with cross laminated timber (CLT) are an attractive alternative to traditional construction materials in terms of environmental performance and habitability, but its structural behavior is not well understood for each timber specie, particularly for the light and soft Chilean radiata pine timber. As the structural response of buildings constructed with CLT elements is controlled by the links between the different members, the study and characterization of the behavior of connections and fasteners in CLT elements made of radiata pine is a must.

Many researches has been developed worldwide aiming to comprehend the behavior of connections in timber structures. Gavric [1] performed an extensive laboratory test programme where the behavior of many different connections and fasteners combinations was assessed, as well as the FPInnovations efforts in Canada [2] and the research of Schneider [3] in Germany, just to mention a few. In Chile, this subject of research is very novel, but Rosales [4] developed a comprehensive laboratory study of hold down connections on radiata pine CLT shear walls.

This work provides in-depth study of the structural behavior of radiata pine CLT shear walls, by means of laboratory testing and numerical analysis of hold down connections. Based on the laboratory results, two hysteretic constitutive laws are calibrated using OpenSees [5] simulations, taking into account the strength and stiffness degradation of the connectors, and the pinching behavior of the load – displacement curves. The hysteretic models considered are Saws model [6] and pinching4 model [7], because both are able to reproduce the observed mechanical behavior of CLT connectors, as it is shown in [8] and [9], although other OpenSees material models can also be used (e. g. ModIMKPinching material and BWBN material).

Main outcomes suggest that advanced modeling tools can accurately reproduce the hysteretic behavior of the connections of timber panels. In terms of connections behavior, it is observed that hold downs on radiata pine CLT elements reach less load capacity than hold downs on other wood specie, and no significant difference with the parallel to grain capacity of angle brackets connections is noticed. Besides, it is found that radiata pine CLT walls can achieve suitable cyclic loading performance and reach high levels of displacement ductility. Furthermore, the importance of friction on the load capacity of the wall is showed.

2. Laboratory test programme

A laboratory test programme regarding to a set of radiata pine CLT walls and hold down connections was performed at Construction Sciences Research Center of the University of Bío Bío (CITEC UBB), and it considered both quasi-static monotonic and cyclic loading testing. The CLT wall samples are 90 mm thick, composed of three layers of C16 [10] grade radiata pine timber of 30 mm thick each one. A Rothoblaas WHT 540 hold down and different fasteners combination are experimentally analyzed. Samples with 2 and 4 LBA Anker nails and samples having 2 and 4 LBS screws were tested under monotonic load, while samples with 12 LBA Anker nails were subjected to cyclic load; both parallel and perpendicular to grain slip, as Table 1 shows.

Table 1: Experimental programme

Connection Type	Hold down	Fasteners	Samples	Load
2C	WHT 540	2 Anker nails	10	Monotonic
2T		2 screws	10	Monotonic
4C		4 Anker nails	10	Monotonic
4T		4 screws	10	Monotonic
12C		12 Anker nails	6 + 12	Cyclic (parallel + perpendicular)

Many different protocols for cyclic testing of timber structures have been developed (e.g. [11]), but in this work the cyclic tests were performed under the EN12512 displacement protocol [11], because it has been widely used in the analysis of CLT elements [13, 14]. This protocol provides a displacement history with three initial cycles of increasing displacement and then four groups of three cycles with the same amplitude.

The setup of the performed cyclic tests is presented on Figure 1. Moreover, monotonic tests were only conducted subjected to tensile load under parallel to grain displacement action. Further information about testing procedures is found on [4].

Figure 1: Parallel to grain hold down cyclic test setup (left) and perpendicular to grain hold down cyclic test setup (right)

3. Numerical modeling and connection tests results

3.1 Monotonic test analysis

As presented on Table 1, a total of 40 samples of different connection types were subjected to experimental test under monotonic parallel to grain displacements. The average monotonic envelope test curves are presented on Figure 2 for every connection type tested (connections 2C, 2T, 4C, and 4T). These mean backbone curves are used to calibrate pinching4 and saws constitutive models in OpenSees [5]. The calibration procedure follow the guidelines proposed on [2], because it is simple, straight forward, and able to reach accurate curve fittings for the connections studied in this work.

Calibrated curves show good agreement with the respective test envelopes, as presented on Figure 2. Both models are able to reproduce the shape of the tested envelope curves, reproducing the rapid increase in strength at low displacement and the subsequent yielding at medium to high displacements. Because of the smoothness nature of saws hysteretic model, the matching is less accurate, particularly for nailed connections (2C and 4C), because the test response exhibits less strength than the model at the displacement range where yielding begins. Moreover, the multi-linear envelope of pinching4 model is capable to reproduce the shape of the curve accurately, notwithstanding that at very low displacements pinching4 backbone curve mismatches the test response. In Table 3 and Table 4 are presented the calibrated parameter for saws and pinching4 hysteretic model respectively.

Results suggest that screwed connections (2T and 4T) perform better than nailed connections (2C and 4C) in terms of load capacity and equivalent elastic stiffness. Connections 2C and 4C are able to reach similar to 2T and 4T F_{max} respectively, but the stiffness degradation is higher for the nailed connections at same load levels. While connection 4T achieves an F_{max} of 14.6 kN and connection 4C of 14.9 kN, the maximum displacement u_{max} for 4T is almost a half of the u_{max} of 4C, being 4.4 mm and 8.5 mm respectively. Nevertheless, nail fastened connections are able to reach higher ductility levels.

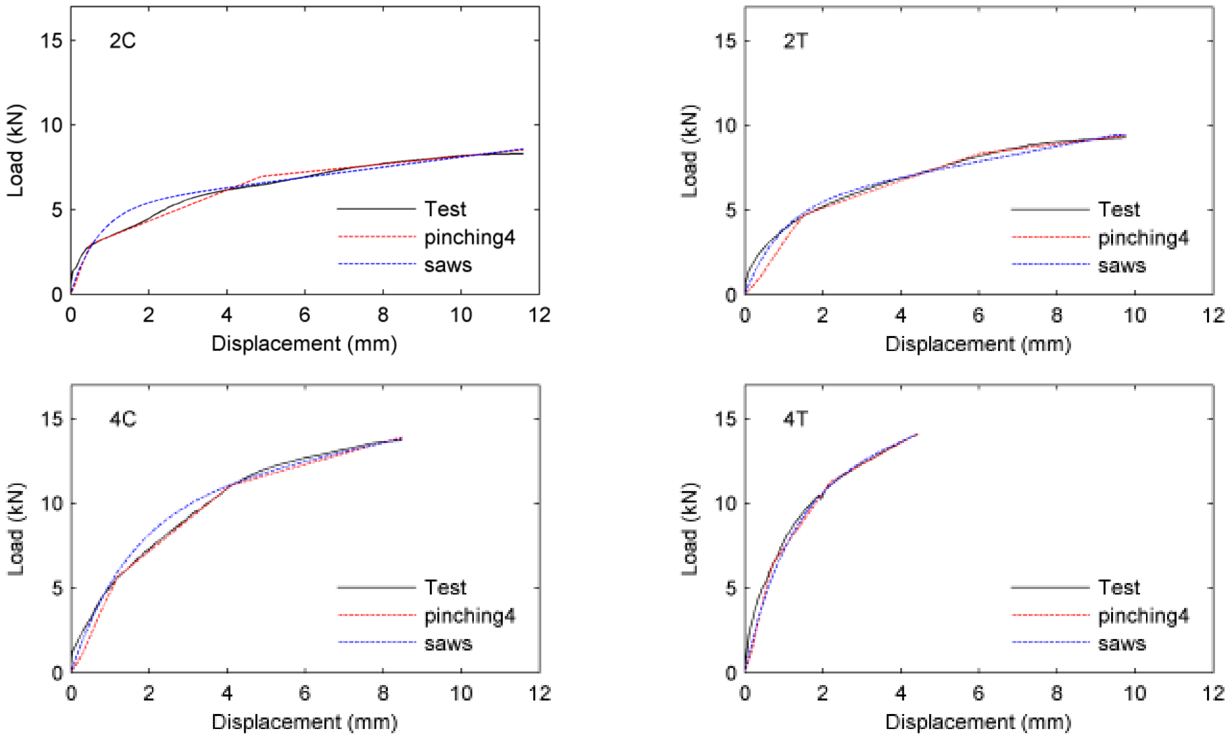

Figure 2: Average monotonic envelope curves of tested connections and pinching4 and saws calibrated responses for 2C, 2T, 4C, and 4T connections.

It is worth noting that the monotonic test results are limited and conditioned by the testing device capacity, because it was designed for a maximum working load of 15 kN Results of connections 4C and 4T are particularly affected by this bounding effect, because they were not able to reach their maximum load and ultimate state during the test.

3.2 Cyclic test analysis

The hysteretic cyclic behavior of the hold down connections on radiata pine CLT elements exhibit important differences between the two analyzed directions of loading (Figure 3 and Figure 4). Parallel to grain tests reach the highest load capacities and equivalent elastic stiffness, of 53 kN and 10 kN/mm respectively, whilst perpendicular to grain tests just achieve around a 20% of these values, due to the twisting of the hold down around the anchor bolt caused by the torsional moment. This effect of large difference on strength and stiffness between the different loading directions is not observed on bracket angle connections, where both directions show similar strength magnitudes [9].

Table 2: Saws model calibrated parameters for parallel to grain monotonic response

Parameter	Connection			
	2C	2T	4C	4T
F_0 (kN)	5.1	5.2	9.7	11.0
D_u (mm)	11.6	9.5	9.0	8.0
S_0 (kN/mm)	7.5	5.9	7.0	10.5
R_1 (mm)	0.04	0.075	0.07	0.07
R_2 (mm)	-0.0002	-0.008	-0.8	-0.8

Table 3: Pinching4 model calibrated parameters for parallel to grain monotonic response

Parameter	Connection			
	2C	2T	4C	4T
ePf1 (kN)	2.9	4.7	5.7	6.4
ePf2 (kN)	6.9	8.3	11.1	11.3
ePf3 (kN)	8.5	9.4	13.9	14.2
ePd1 (mm)	0.5	1.5	1.2	0.7
ePd2 (mm)	4.9	6.0	4.1	2.2
ePd3 (mm)	11.6	9.8	8.5	4.5
eNf1 (kN)	-2.9	-4.7	-5.7	-6.4
eNf2 (kN)	-6.9	-8.3	-11.1	-11.3
eNf3 (kN)	-8.5	-9.4	-13.9	-14.2
eNd1 (mm)	-0.5	-1.5	-1.2	-0.7
eNd2 (mm)	-4.9	-6.0	-4.1	-2.2
eNd3 (mm)	-11.6	-9.8	-8.5	-4.5

For perpendicular to grain hold down connections testing a pinched hysteresis with strength hardening is observed (Figure 3). Unloading path is almost vertical until the load reaches zero and then is horizontal until the displacements achieve its central value, from where the loading loop begins to increase the strength. Since tests were not conducted at high levels of displacement, the possible decay and degradation of the strength is not observed.

Moreover, parallel to grain hysteretic curves exhibit a quick strength drop after reaching the ultimate capacity, with a rupture displacement of 42 mm (Figure 4). Besides, the ultimate strength is 53 kN at a parallel to grain displacement of 17 mm. As the yielding displacement is about 10 mm (at a restoring force of 48 kN), the displacement ductility achieved is 4.2 at the failure of the connection.

The strength exhibited by the tested connections is just about a 40% of the strength reached in other researches for similar connections and equivalent number and type of fasteners [9]. Notwithstanding this difference, the yielding displacement and displacement at ultimate strength are almost equal. Another noticeable difference is regarded with the strength degradation after ultimate displacement, since the connections tested in this study reduce the strength much more quick. For example, at a same displacement ductility of 3.5, connections analyzed in this work exhibit a strength of a 22% of the ultimate value, whilst for connections presented on [9] the resistance is about a 72% of the maximum strength.

Based on test results, pinching4 and saws hysteretic models are calibrated on OpenSees [5] by adjusting the parameters of each model, giving Table 4 and Table 5 values. The calibration procedure follow the guidelines proposed on [2], which is a reverse calibration procedure, based on an iterative sequence of adjusting the model parameters until the model closely match the observed response.

Results show good fitting between the simulated and the measured force - displacement curves for every connection tested and for both constitutive laws, but pinching4 model exhibits better adjustment than Saws model, particularly for the cyclic perpendicular to grain tests, as it is observed on Figure 3, due to the capability of the model to accommodate the constricted shape of the pinched hysteresis response, effect which poorly replicated by saws on this tests, thus it suggests that saws may over damp the response of the connections in terms of hysteretic energy dissipation when subjected to perpendicular to grain loads. Furthermore, pinching4 model is able of a better reproduction of the strength hardening of the connection subjected to perpendicular to grain test, as well as the unloading path.

Regarding the accuracy of models to adjust to the cyclic response of the parallel to grain test, Figure 4 show that the envelope is better reproduced by saws, but the loading – unloading path is better agreed by pinchig4, due to its capability to properly adjust the reloading an unloading stiffness.

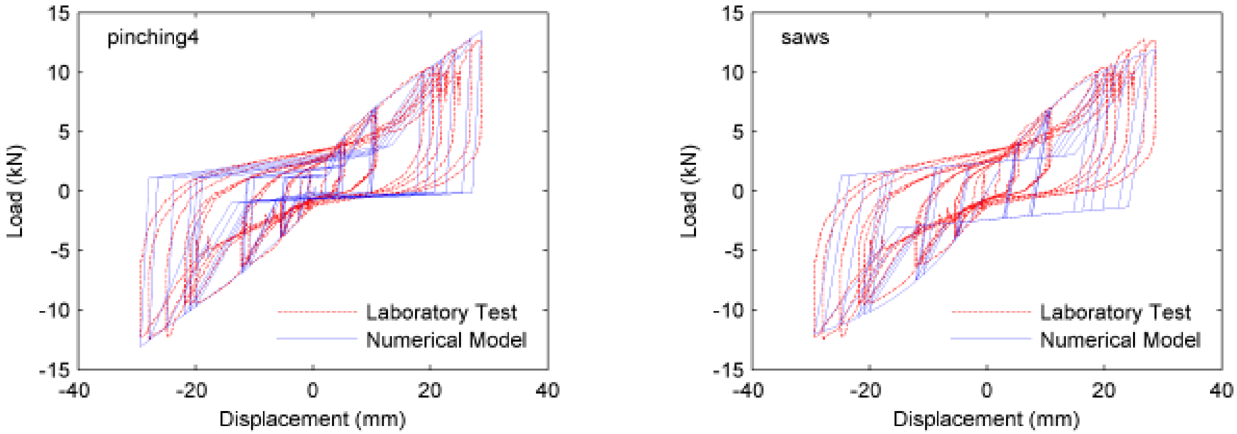

Figure 3: Hysteretic response of a hold down connection subjected to perpendicular to grain cyclic test, pinching4 model (left) and saws model (right).

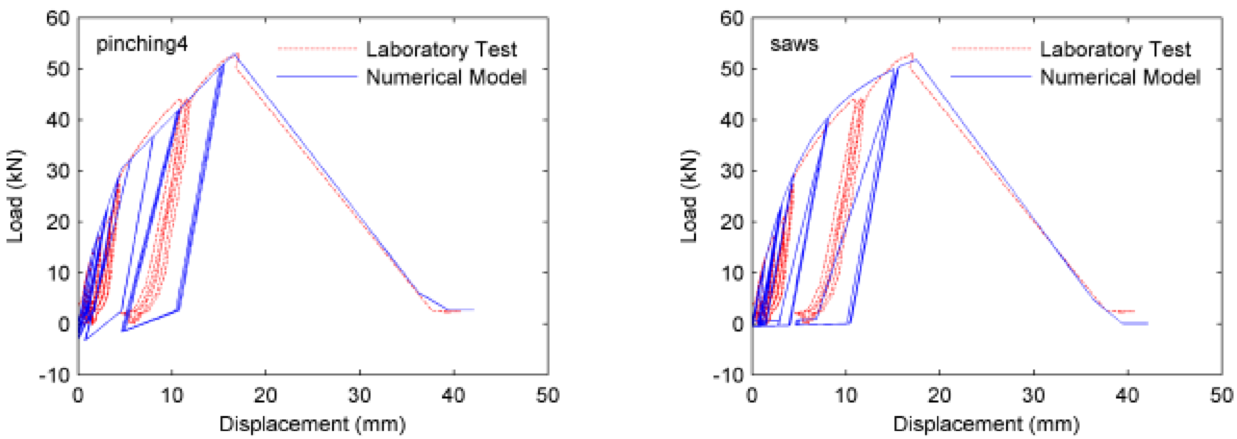

Figure 4: Hysteretic response of a hold down connection subjected to parallel to grain cyclic test, pinching4 model (left) and saws model (right).

Table 4: Saws model calibrated parameters for cyclic response of connection 12C

Parameter	Direction		Parameter	Direction	
	Perpendicular	Parallel		Perpendicular	Parallel
F_0 (kN)	13.0	50.0	R_2 (mm)	-0.005	-0.3
F_1 (kN)	2.4	5.0	R_3 (mm)	3.2	0.95
D_u (mm)	30.0	14.5	R_4 (mm)	0.05	0.015
S_0 (kN/mm)	0.9	6.5	α	0.45	0.45
R_1 (mm)	0.03	0.005	β	1.03	1.03

4. Connection model validation: full scale wall test analysis

A full scale 2,95 x 2.95 m radiata pine CLT shear wall supported over 2 hold downs and 2 shear brackets is subjected to a cyclic test under the EN12512 horizontal displacement protocol, reaching a lateral drift of almost 1%.

The CLT wall prototype is composed by 3 layers of radiata pine timber, with an overall thick of 90 mm. A vertical load of 18 kN/m is applied on the top of the wall. The experimental test was conducted at the Structural Engineering Laboratory of CITEC UBB. Further details about testing procedures and equipment can be found on [4]. The test results show that the CLT panel rotates almost as a rigid body where its kinematic response is controlled by the displacement behavior of the supports. Also, the nonlinear response of the connectors provides the ductility and energy dissipation for the wall under the cyclic load. The maximum lateral displacement ductility achieved during the wall test is around 2.5.

Table 5: Pinching4 model calibrated parameters for cyclic response of connection 12C

Parameter	Direction		Parameter		
	Perpendicular	Parallel		Perpendicular	Parallel
ePf1 (kN)	2.6	9.9	fForceN	0.3	0.4
ePf2 (kN)	6.9	30.3	uForceN	0.08	-0.05
ePf3 (kN)	13.1	43.8	gK1	-2.5	-2.5
ePf4 (kN)	1.5	12.7	gK2	0.0	0.0
ePd1 (mm)	2.5	2.1	gK3	0.0	0.0
ePd2 (mm)	12.1	8.2	gK4	0.0	0.0
ePd3 (mm)	29.6	14.0	gKLim	-8.5	-0.5
ePd4 (mm)	70.0	35.0	gD1	0.0	0.0
eNf1 (kN)	-2.6	-9.9	gD2	0.0	0.0
eNf2 (kN)	-6.9	-30.3	gD3	0.0	0.0
eNf3 (kN)	-13.1	-43.8	gD4	0.0	0.0
eNf4 (kN)	-1.5	-12.7	gDLim	0.0	0.08
eNd1 (mm)	-2.5	-2.1	gF1	0.0	0.0
eNd2 (mm)	-12.1	-8.2	gF2	0.0	0.0
eNd3 (mm)	-29.6	-14.0	gF3	0.0	0.0
eNd4 (mm)	-70.0	-35.0	gF4	0.0	0.0
rDispP	0.5	0.5	gFLim	0.0	0.0
fForceP	0.08	0.3	gE	1	1
uForceP	0.01	-0.05	Damage type	Energy	Energy
rDispN	0.5	0.5			

In terms of damage, the CLT tested wall suffers almost no damage. Only the bottom corners of the wall, which are in direct contact with the foundation element suffers noticeable damage caused by local crushing of the timber edges. Another source of damage is the wood crushing at fasteners on the connections (Figure 5 right), which damage level can be classified as minor damage in accordance to the scale proposed on [15] for CLT connections.

Using the calibrated numerical models of the hold down connectors, the cyclic behavior of the tested wall is simulated numerically. As angle brackets connections were not tested, the parameters for constitutive models are obtained from [8], even though they were calibrated for other timber specie and grade, other connector manufacturer, and other fastener type. The wall is modeled by isotropic elastic shells with a Young Modulus of 9000 MPa [10]. Supports are idealized coupling orthogonal springs by means of tridimensional zeroLength elements fixed at one end, and the other is connected with the wall nodes. Additionally, the interaction of the wall with the foundation and vertical load device is modeled with contact elements (zeroLengthContact3D OpenSees elements) aiming to take into account the sliding and uplifting movement of the wall under rocking motion as well as the frictional resistant force of the system. Due to numerical issues, a layer of auxiliary dummy nodes is created in order to connect contact elements with wall and foundation nodes.

4.1 Kinematic accuracy

The fidelity of the model to reproduce the rocking motion of the wall during the test is revised by a comparison of the displacements at wall corners. On Figure 6 is presented the displacements time histories of the vertical uplift at both sides of the wall and the horizontal slip at the bottom of the wall. Good matching between the experimental and simulated displacement time series is observed until the 10th loading cycle at the second 2150 of the analysis (signed as the plotted vertical purple line). Since this instant onwards, large differences are obtained for every studied movement, effect explained by a failure on the testing device which generates an important decrease on the vertical load applied over the wall, leading to an accumulation of sliding displacement on the positive direction of analysis, bearing to an increment of the uplifting at the left corner of the wall and a reduction of vertical displacement at the opposite corner. This effect is poorly replicated by the model, because the magnitude of the vertical force loss was not possible to measure, as well as the precise instant and wall state when the malfunction occurs. Thus, this effect was not feasible to accurately include in the model.

Figure 5: Tested wall supports layout (left). Timber crushing on hold down fasteners holes (right)

Notwithstanding the above mentioned phenomenon, conclusions can be made for the part of the test on where the testing device performed well. The kinematic response and rigid body rotation about the bottom corners of the wall is well captured by the model. Good fitting is observed for the horizontal slip at bottom of the wall and at right corner vertical uplift, where the differences between the model and the test response is around a half of millimeter and one millimeter for vertical and horizontal displacements respectively, giving discrepancies of less than 15% in terms of the maximum displacement per each loading cycle.

For the left corner vertical displacements, higher differences are observed, particularly for displacements when compressive forces are acting on the edge and the vertical downwards displacement is restrained by the foundation. These differences can be explained by the crushing of the timber in the corner of the wall and its consequent increase in the vertical descent of the wall caused plastic deformation concentration. This phenomenon is not reproduced by the model since it considered elastic constitutive material for the wall elements.

4.2 Force response accuracy

Overall good fitting between the measured lateral force resultant on the experimental test and the horizontal force obtained in the model is reached, but again the malfunction of the testing device lead to wrong model prediction since the second 2150 onwards (see Figure 7). The resistant force of the model is determine as the sum of the horizontal force at angle bracket and hold down connections added with the friction force at the contact interface.

Figure 7 force time series show that friction supplies a large proportion of the lateral resistant force of the model, being almost a 60% in certain cycles, hinting that under particular loading conditions, the response of the wall is controlled more by the friction effects than the connection horizontal force itself. This effect can be particularly more critical on the response of heavily compressive loaded CLT elements. On the other hand, in CLT elements under low compressive loads, the capacity could be determined only by the force provided by the connections.

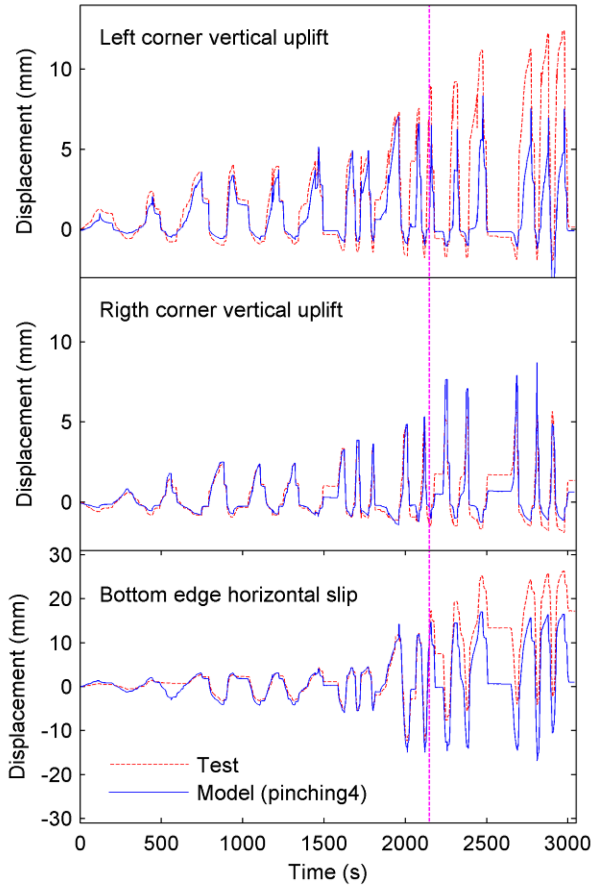

Figure 6: Kinematic response of the wall during cyclic test and modelling. Bottom left wall corner vertical displacement time series (top); bottom right wall corner vertical displacement time series (middle); and wall bottom horizontal slip time history (bottom).

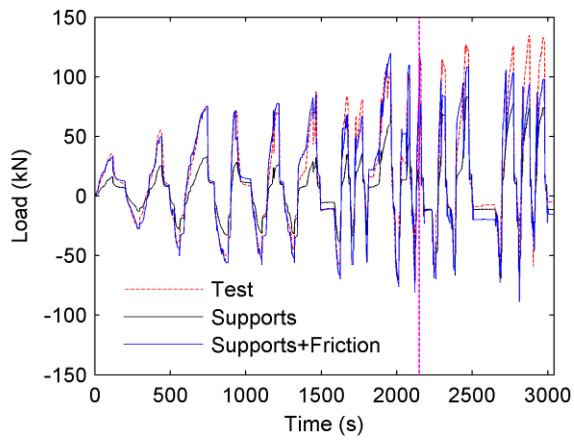

Figure 7: Lateral load time history of the wall during the test and simulated support and friction horizontal load.

Regarding the lateral force magnitudes, during the test the wall achieve a maximum load of 135 kN at the lasts loading cycles, on where the vertical load was diminishing. On the part of the test with controlled vertical load, both test and model reach a lateral load about 110 kN, divided on 58 kN provided by supports and 52 kN supplied by friction.

4.3 Hysteresis behavior accuracy

Regarding the load – displacement behavior of the wall, numerical analysis are able to reproduce the general shape of the hysteresis curves, but show some pitfalls, as it can be seen on Figure 9.

On Figure 9 are presented the results of the wall test simulation with connections modeled using the calibrated pinching4 constitutive law, showing the total horizontal load vs. top lateral displacement, bottom lateral displacement, and right side vertical uplift. On the lateral load – top lateral displacement plot (Figure 9 – a – left) is observed that the wall model is capable to match the positive loading loops in terms of cycle shape, and maximum load and displacement. Besides, at the negative loading stages the fitting becomes coarse, giving poor similitude particularly at unloading path, on where the model tends to overestimate the drop of the lateral force.

For the displacement at bottom of wall (Figure 9 – b – left), the asymmetric displacement behavior is barely reproduced. When the wall is pushed (positive displacements), model achieves less displacements, while when is pulled, simulation brings more displacements. Although this effect, the differences are less than 15% of the measured values.

Regarding the uplifting response presented on Figure 9 – c – left, a bilinear behavior is observed, where the model is stiffer than the test for upward movement, but it is softer for the downward displacement. This result suggests that the contact interface used to idealize the vertical displacement restraint at the foundation should be improved in order to reach better results.

For the wall analyzed with calibrated saws model supports, the likeness with the tested results is poorest. A comparison between the measured and modeled wall hysteretic response with saws idealized connections is presented on Figure 9 – right. A coarser than pinching4 modeled connections fitting is observed for every analyzed displacement, in particular for the top horizontal displacement. It is also noticed that saws modeled connections tend to overestimate the uplifting of the wall, but underestimate the lateral sliding at the bottom of the wall. This effect is explained by the lack of likeness between the pinched zone and the strength drop of the test results and the calibrated saws model for the hold down subjected to perpendicular to grain load (Figure 3 right).

4.4 Wall capacity at connection rupture state

Using a simulation, the maximum response of the wall subjected to a monotonic load which reaches the maximum displacement capacity of the pulled hold down is explored. For this analysis, the wall modeling approach is the same used in the connection validation analysis, as well as the distributed vertical load of 18 kN/m applied on the top of the wall. An ultimate capacity of the wall of 154 kN at a drift level of 1.4% is observed on Figure 8, giving a displacement ductility of 4.6. After the maximum load, the capacity of the wall falls until a load of 108 kN, providing suitable displacement capacity and ductile behavior (maximum displacement of 74 mm with a displacement ductility of 7.4). Moreover, the total lateral capacity of the wall is divided into the force supplied by the supports (hold downs and angle brackets) and the force provided by the friction between the wall and the foundation in almost equal parts. Besides, the spikes and abrupt load drop in the backbone curve are caused by the node-to-node behavior of the contact interface used.

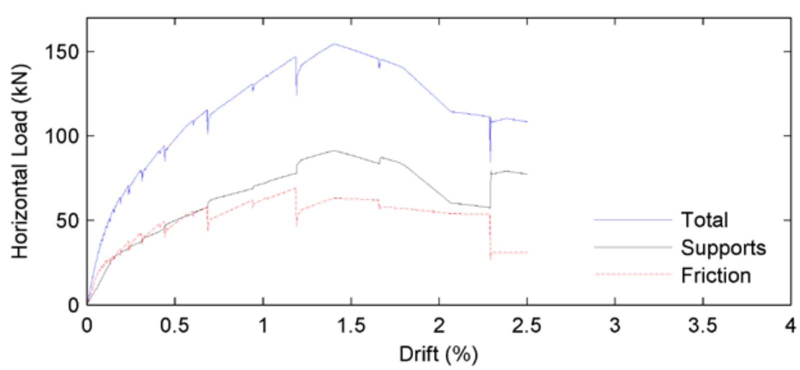

Figure 8: CLT wall capacity curve

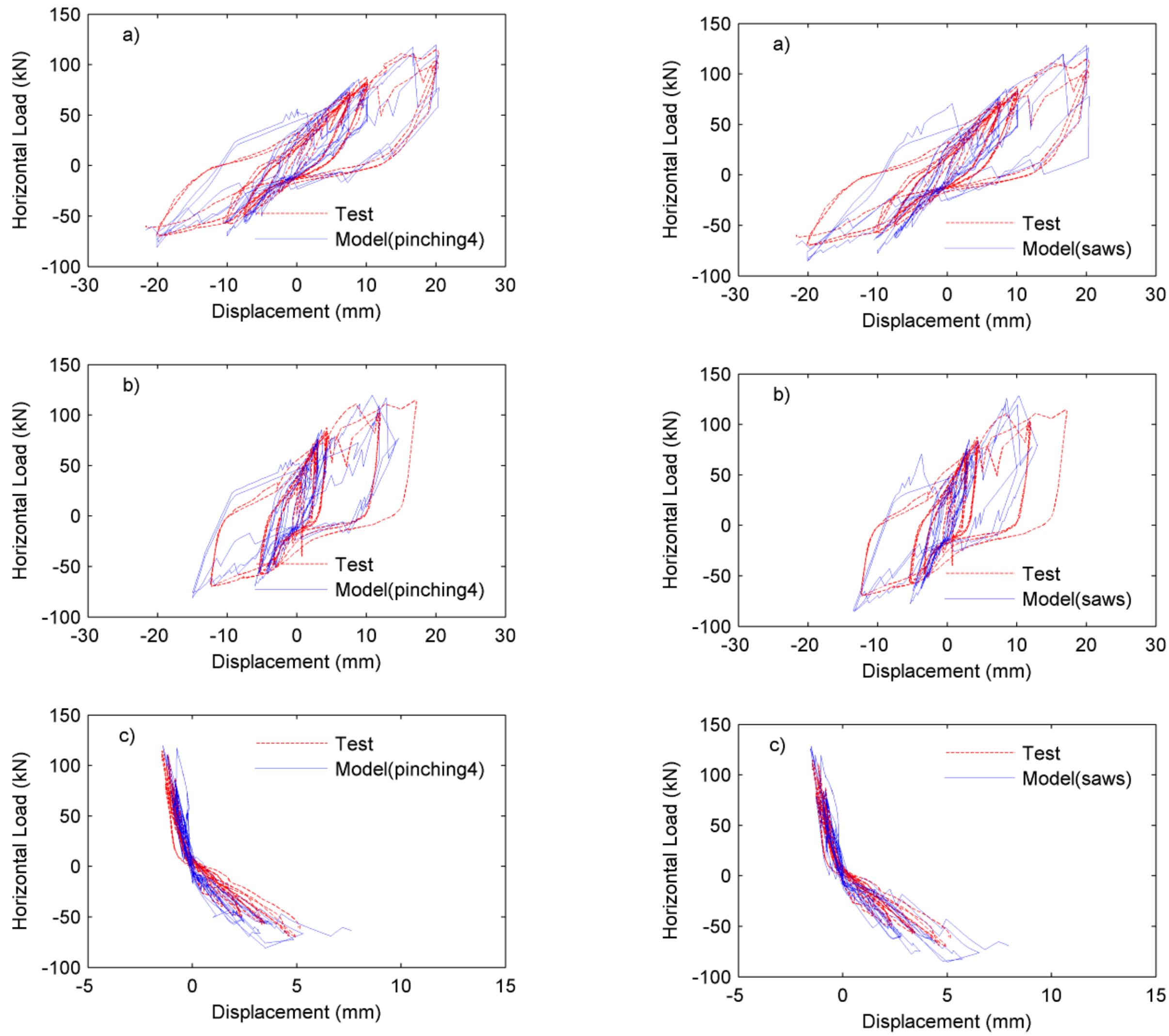

Figure 9: Observed and modeled hysteretic response of tested wall (left for pinching4 model and right for saws model). a) Lateral load v/s wall top horizontal displacement. b) Lateral load v/s wall bottom horizontal displacement. c) Lateral load v/s bottom right wall corner vertical displacement.

5. Conclusions

A set of experimental and numerical analyses is done on different connections and fasteners for a radiata pine CLT shear wall. Results show that the hysteretic models used to replicate the cyclic response of the hold down connection are capable to reproduce the observed load – displacement behavior for either monotonic or cyclic analysis of connections, as well as the response of a full scale wall, despite all the simplifications done in the modeling. This result appears to show that the behavior of these structural elements can also be explored numerically with confidence.

In general terms, the better agreement between connection tests and model results is observed for pinching4 model, which is explained by the flexibility of the model given by the large number of parameters that control the response, providing better accommodation to complex hysteretic behaviors. However, more calibration effort is needed. Nevertheless, saws model exhibit also good fitting between test and model results at connection level, but at wall level, it overestimate the lateral capacity and the uplifting movement, and underestimate the sliding between the wall and the foundation.

Regarding the influence of the wood specie, hold down connection in radiata pine CLT reaches less load capacity than the one achieved for other timber species reported on the literature.

Simulation results suggest that the friction force shall not be oversimplified or discarded in the analysis or design of CLT structures, particularly when the elements are subjected to high compressive loads, which may lead to high friction forces between elements. Further research is needed.

Furthermore, the research suggests that CLT shear walls can reach significant displacement ductility levels, implying suitable performance for seismic loading. The simulation results show that the CLT wall is able to achieve displacement ductility over 7 with a drift capacity of almost 2.5%.

6. References

[1] Gavric I (2012): *Seismic Behavior of Cross-Laminated Timber Building* (PhD Thesis). Università Degli Studi di Trieste, Trieste, Italy.

[2] Karacabeyli E, Douglas B (2013): *Cross - Laminated Timber Handbook. US Edition*. Point-Claire, Canada: FPInnovations.

[3] Schneider J (2009): *Connections in cross-laminated timber shear walls considering the behavior under monotonic and cyclic lateral loading* (MSc Thesis). University of Stuttgart, Stuttgart, Germany.

[4] Rosales V (2016): *Comportamiento de unión fundación – muro en aplicación estructural de Madera Contralaminada de Pino Radiata* (MSc Thesis). Universidad del Bío-Bío, Concepción, Chile.

[5] McKenna F, Fenves G, Filippou F, Mazzoni S, Scott M, Elgamal A, Yang Z, Lu J, Arduino P, McKenzie P, Deierlein G, Law K (2006): *OpenSees.* University of California, Berkeley, USA.

[6] Folz B, Filiatrault A (2001): SAWS - Version 1.0, A Computer Program for the Seismic Analysis of Woodframe Structures. Structural Systems. *Research Project Report SSRP-2001/09*. University of California, San Diego, USA.

[7] Lowes L, Mitra N, Altoontash A (2004): A beam-column joint model for simulating the earthquake response of reinforced concrete frames. *Technical Report PEER 2003/10*. Pacific Earthquake Engineering Research, Berkeley, USA.

[8] Shen YL, Schneider J, Tesfamariam S, Stiemer SF, Mu ZG (2013): Hysteresis behavior of bracket connection in Cross-Laminated-Timber shear walls. *Construction and Building Materials*, 48(2013): 980–991.

[9] Liu J, Lam K (2014): Numerical simulation for the seismic behaviour of mid-rise CLT shear walls with coupling beams. *2014 World Conference on Timber Engineering*, Quebec, Canada.

[10] EN 338:2009 (2009): Structural timber. Strength classes.

[11] Krawinkler H, Parisi F, Ibarra L, Ayoub A, Medina R (1999): Development of a testing protocol for woodframe structures. *W-02. CUREE Publication*. Consortium of Universities for Research in Earthquake Engineering. Richmond, USA.

[12] EN 12512:2001 (2001): Timber structures – test methods – cyclic testing of joints made with mechanical fasteners.

[13] Gavric I, Fragiacomo G, Ceccotti A (2012): Strength and deformation characteristics of typical X-LAM connections. *2012 World Conference on Timber Engineering*, Auckland, New Zealand.

[14] Gavric I, Rinaldin G, Amadio C, Fragiacomo M, Ceccotti A (2012): Experimental-numerical analyses of the seismic behavior of Cross-Laminated wall systems. *15th World Conference of Earthquake Engineering*, Lisbon, Portugal.

[15] Schneider J, Karacabeyli E, Popovski M, Stiemer SF, Tesfamariam S (2014): Damage assessment of connections used in Cross-Laminated Timber subject to cyclic loads. *Journal of Performance of Constructed Facilities*, 28(6): A4014008.