Seismic Behaviour of Cross-Laminated Timber Structures

Marjan Popovski
Principal Scientist, Advanced Building Systems, Fpinnovations, , Vancouver, BC

Erol Karacabeyli
Manager, Advanced Building Systems, Fpinnovations, Vancouver, BC

15 WCEE
LISBOA 2012

SUMMARY
FPInnovations has undertaken a multi-disciplinary project on determining the structural properties of a typical CLT construction. One of the important parts of the project is quantifying the seismic resistance of structures with CLT panels. In this paper, results from a series of quasi-static monotonic and cyclic tests on CLT wall panels are presented and discussed. CLT wall panels with various configurations and connection details were tested. Wall configurations include single panels without openings with three different aspect ratios, panels with openings, as well as multi-panel walls with step joints and fasteners between them. Connections for securing the walls to the foundation include off-the-shelf steel brackets with annular ring nails, spiral nails, and screws, a combination of steel brackets and hold-downs, and custom made brackets with timber rivets. In addition, results from two storey configurations that include two walls and a CLT slab in between are presented and discussed. Finally, some recommendations for the force modification factors (R-factors) for seismic design of CLT structures according to National Building Code of Canada are made.

Keywords: Cross Laminated Timber, Seismic Performance, R-factors

1. INTRODUCTION AND BACKGROUND

Cross-laminated timber (CLT) was first developed some 20 years ago in Austria and Germany. European experience shows that this system can be competitive not only in low rise but also in mid-rise and high-rise buildings. Although CLT has barely being used in North America so far, it may be used as a viable wood-based structural solution for the shift towards sustainable densification of urban and suburban centres in Canada and the US. In order to gain wide acceptance, CLT as a structural system needs to be implemented in the North American building codes arena. For these reasons FPInnovations has undertaken a multi-disciplinary research project on determining the structural properties of CLT construction. One of the important parts of the project is to quantify the seismic behaviour of CLT structures including the development of the force modification factors (R-factors) for seismic design according to National Building Code of Canada (NBCC). In this paper some of the results from a series of quasi-static tests on CLT wall panels are presented along with preliminary analyses of R-factors.

2. PREVIOUS RESEARCH IN THE FIELD

The most robust study to date to quantify the seismic behaviour of CLT construction was the SOFIE project undertaken by the Trees and Timber Institute of Italy (CNR-IVALSA) in collaboration with Japanese researchers from NIED, Shizuoka University, and the BRI. The testing programme included in-plane cyclic tests on CLT wall panels with different layouts of connections and openings (Ceccotti et. al. 2006b), pseudo-dynamic tests on a one-storey 3-D specimen in three different layouts, and shaking table tests on a three-storey (Ceccotti and Folessa, 2006) and seven-storey CLT building. Shaking table tests on the 3-storey building showed that the CLT construction survived 15 destructive earthquakes without any severe damage even at peak ground accelerations (PGA) of 1.2g (Ceccotti, 2008). Similarly, the 7-storey building was able to withstand strong earthquakes such as the Kobe one without any significant damage. A comprehensive study to determine the seismic behaviour of 2-D CLT wall panels was conducted at the University of Ljubljana. Numerous monotonic and cyclic tests were carried out on walls with different aspect ratios and boundary conditions from the cantilever type all the way to the pure shear (Dujic et. al. 2004). Influence of vertical load and type of anchoring systems were evaluated, along with wall deformation mechanisms (Dujic and Zarnic, 2006). In addition, influence of openings on the shear properties of CLT wall panels was studied and formulae were suggested (Dijic et. al. 2007). Shaking table tests were also conducted on two single story box CLT models at IZIIS in Skopje, Macedonia (Dujic et. al. 2006). Finally, a blind analytical prediction of the response of the 7-storey CLT structure tested during the SOFIE project was conducted (Dujic et. al. 2010).

CLT wall tests were also carried out by the Karlsruhe Institute of Technology to compare the performance of such modern system vs. the traditional timber frame construction (Schädle and Blaß, 2010). Analytical models of a three-storey 2-D CLT structure (frame) was developed in DRAIN-2DX and a series of 20 earthquake records was used to determine the behaviour factor q according to Eurocode 8. The average value of q-factor obtained with slightly modified PGA approach was 4.7, with the 5^{th} percentile value being 3.3 for all 20 earthquakes. Upon completion of the SOFIE project, further research was carried out in Italy to determine the hysteretic behaviour of single CLT walls, bracket connections, wall half-lap connections with different fasteners, and hold-downs (Gavric et al. 2011). Analytical models for connections in CLT structures (brackets, hold-downs and connections between panels) were developed in Abacus software (Fragiacomo and Rinaldin 2011) and the models were used to develop CLT wall models. In addition, elastic and ductile design approaches for multi-storey CLT buildings under seismic loads were developed (Fragiacomo et al. 2011).

3. CLT WALL SPECIMENS TESTED AND THE TEST SETUP

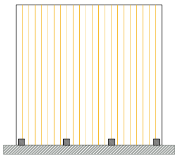
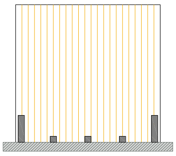
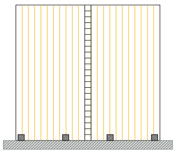
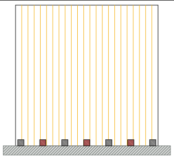
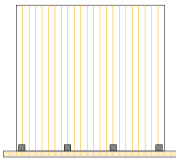
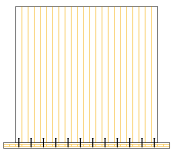
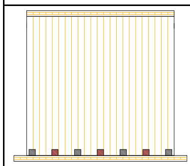
A total of 32 monotonic (ramp) and cyclic tests were performed on 12 configurations of CLT walls. The testing matrix that includes wall configurations I to IX is given in Tables 1 and 2. Three additional wall configurations (walls with openings, 4.9 m tall walls, and two-storey CLT assemblies) were also tested but these results are not reported in this paper, but reported in (Popovski et. al. 2011). All walls were 3-ply CLT panels with a thickness of 94 mm, made of European spruce and manufactured at KLH Massiveholz GmbH in Austria. Three different types of brackets (A, B, and C) were used to connect the walls to the steel foundation beam or to the CLT floor panel bellow (Figure 1). Bracket A (BMF 90 mm x 48mm x 116 mm, and bracket B (Simpson Strong Tie 90 mm x 105 mm x 105 mm) were off-the-shelf products that are commonly used in CLT applications in Europe, while bracket C was custom made out of 6.4mm thick steel plates to accommodate the use of timber rivets.

a) Bracket A b) Bracket B c) Bracket C

Figure 1. Brackets for CLT walls used in the tests

The designations of the tests shown in Tables 1 and 2 were developed to show the bracket type and the fastener type used. For example designation CA-SNH-08A (from configuration II) means that the **C**LT wall has brackets type **A**, **S**piral **N**ails as fasteners, has **H**old-downs and it is test number **08A**. The following acronyms were also used: **TR** for Timber Rivets, **RN** for Annular Ring Nails, **S1** for SFS screws 4 x 70mm, **S2** for SFS screws 5x90mm, and **WT** for SFS WT-T type screws.

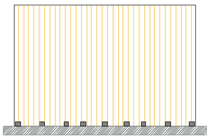
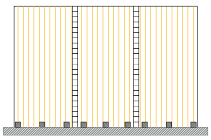
Table 1. Test matrix for 2.3m long and 2.3m high CLT walls (aspect ratio 1:1)

Wall Configuration	Test Designation	Brackets and Fasteners	Vertical Load [kN/m]	Lateral Load
 I	CA-SN-00	Bracket A placed at 710mm SN 16d, n = 18 D = 3.9 mm L = 89 mm	0	CUREE
	CA-SN-01		10	Ramp
	CA-SN-02		10	CUREE
	CA-SN-03		20	CUREE
	CA-RN-04	RN 10d (3.4 x 76mm), n=12	20	CUREE
	CA-S1-05	S1 (4 x 70mm) n=18	20	CUREE
	CA-S2-06	S2 (5 x 90mm) n=10	20	CUREE
	CC-TR-09	Bracket C, Rivets L=65mm, n=10	20	Ramp
	CC-TR-10A	Bracket C, Rivets L=65mm, n=10	20	CUREE
 II	CA-SNH-07	SN 16d (3.9 x 89mm), n=18 Same on Hold-Down	20	Ramp
	CA-SNH-08	SN 16d (3.9 x 89mm), n=18 Same on Hold-Down	20	CUREE
	CA-SNH-08A	SN 16d (3.9 x 89mm), n=18 12d (3.3 x 63mm), n=18 on HD	20	CUREE
 III	CA-SN-11	SN 16d (3.9 x 89mm), n=18 WT-T (3.8 x 89mm) n=12	20	CUREE
	CA-SN-12	SN 16d (3.9 x 89mm), n=18 SFS2 (5 x 90mm) n=12	20	CUREE
	CA-SN-12A	SN 16d (3.9 x 89mm), n=18 Between panels SFS2 (5 x 90mm) n=12	20	ISO
 IV	CA-SN-20	Bracket A SN 16d , n=18 D=3.9mm L=89mm 3 brackets on the back side Representative of inside bottom storey wall	20	CUREE
 V	CA-SN-21	Bracket A at 710mm SN 16d, n=6 D = 3.9 mm L = 89 mm Representative of upper storey wall	20	CUREE
 VI	CS-WT-22	WTT-T n=18 at D = 6.5 mm L = 130 mm Screws spaced at 280 mm	20	CUREE
	CS-WT-22B	WTT-T n=34 D = 6.5 mm L = 130 mm Screws spaced at 40mm/320 mm	20	CUREE
 VII	CA-SN-23	Bracket A SN 16d, n=6 D=3.9mm L=89mm 3 brackets on the back side Representative of inside wall	20	CUREE

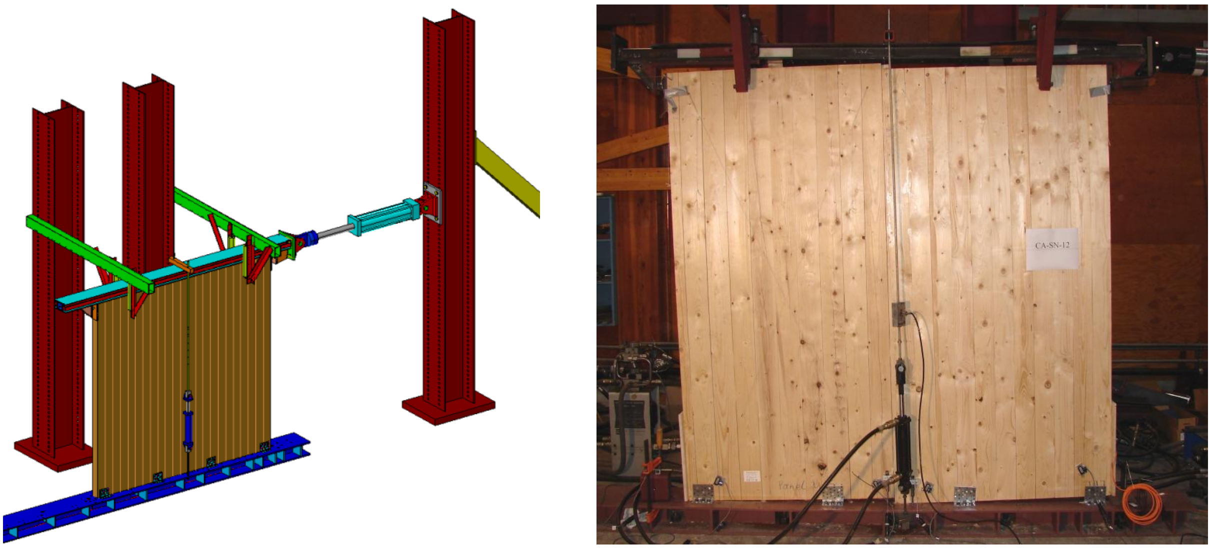
Walls from configurations I were single panel CLT walls used at the bottom storey of a structure (connected to a steel foundation) with various fasteners in the brackets. In addition to that, walls from configuration II had hold-downs at both ends of the wall. Walls from configuration III (11, 12 and 12A) consisted of two panels that were connected to each other using a continuous 65 mm step-joint with no gap, and one vertical row of screws. Twelve SFS WTT-T type screws 3.8 mm x 89 mm, spaced at 200 mm were used in the step-joint in wall 11, while walls 12 and 12A used 5 mm x 90 mm

SFS screws. To investigate the effect of the foundation stiffness (walls in upper storeys), walls in configurations V, VI and VII were placed over a 94mm thick CLT slab with a width of 400mm. The brackets were connected to the CLT floor slab using 3 SFS WFC screws (D=10mm and L=80 mm).

Table 2. Test matrix for 3.45m long and 2.3m high CLT walls (aspect ratio 1:1.5)

Wall Configuration	Test Designation	Brackets and Fasteners	Vertical Load [kN/m]	Lateral Load
 VIII	CB-SN-13	Bracket B SN 16d (3.9 x 89mm) n=10 9 brackets	20	Ramp
	CB-SN-14	Bracket B SN 16d (3.9 x 89mm), n=10 9 brackets	20	ISO
 IX	CB-SN-15	Bracket B SN 16d (3.9 x 89mm) n=10 9 brackets; SFS2 (5 x 80mm) n=8	20	Ramp
	CB-SN-16	Bracket B SN 16d (3.9 x 89mm), n=10 9 brackets SFS2 (5 x 90mm) n=8	20	ISO

Wall 22 was connected to the floor slab using 9 pairs of SFS WT-T 6.5 mm x 130 mm screws placed at 45 degree angle to the slab. Wall 22B used 17 pairs of the same screws with 5 pairs being closely grouped near each end of the wall (spaced at 40 mm) to simulate a hold-down effect. The rest of the screws were spaced at 320 mm. Walls 13 and 14 were single panel walls with a total of 9, type B brackets spaced from 320 mm to 460 mm. Walls 15 and 16 of configuration IX were 3-panel walls, with the same number and position of the brackets as the walls of configuration VIII. The panels were connected between them by step joints and 8 SFS 5x90mm screws spaced at 300mm.



a) b)

Figure 3. a) Sketch of the test setup used for CLT walls; b) CLT Wall 12 during testing

A sketch of the test set-up with a specimen ready for testing is shown in Figure 3a. Steel I beam provided a foundation to which the specimens were bolted down. Another stiff steel beam that was bolted to the CLT walls was used as lateral load spreader bar. Lateral guides with rollers were also used to ensure a steady and consistent unidirectional movement of the walls. Vertical load was applied using a single 13.3 kN hydraulic actuator placed in the middle of the wall in case of 2.3 m long walls (Figure 3a), or with two such actuators located at third points in case of 3.45 m long walls. Walls 01 and 02 were tested with 10 kN/m vertical load that approximately corresponds to a wall being at the bottom of a two storey structure. All other walls (except wall 00 with no load) were tested using 20 kN/m vertical load that corresponds to a wall being at the bottom of a four storey structure. The walls were subjected to either monotonic or cyclic lateral loading. The rate of loading for the monotonic (ramp) tests was 0.2 mm/s. Cyclic loading tests were carried using either CUREE (Method C) or ISO 16670 (Method B) testing protocols given in ASTM E2126 (ASTM, 2009), with a rate of 5mm/s.

4. RESULTS AND DISCUSSION

As expected, the CLT wall panels behaved almost as rigid bodies during the testing. Although slight shear deformations in the panels were measured, most of the panel deflections occurred as a result of the deformation in the joints. In case of multi-panel walls, deformations in the step joints also had significant contribution to the overall wall deflection. Selected average properties (based on both sides of the hysteretic loops) of the CLT wall tests obtained from the experimental program are given in (Popovski et. al. 2011). The value of axial load had an impact on the lateral resistance of the walls. Wall 00 with no vertical load reached a maximum load of 88.9 kN while wall 02 with 10kN/m vertical load had a lateral resistance of 90.3 kN. When the vertical load was increased to 20kN/m (wall 03) the resistance increased to 98.1 kN, an increase of 10% (Figure 4). It seems that the axial load had to be 20kN/m or higher to have any significant influence on the lateral load resistance. The amount of vertical load, however, had a higher influence of the wall stiffness. The stiffness of wall 03 was 28% higher than that of wall 00. In addition, higher values of vertical load had influence on the shape of the hysteresis loop near the origin (Figure 4).

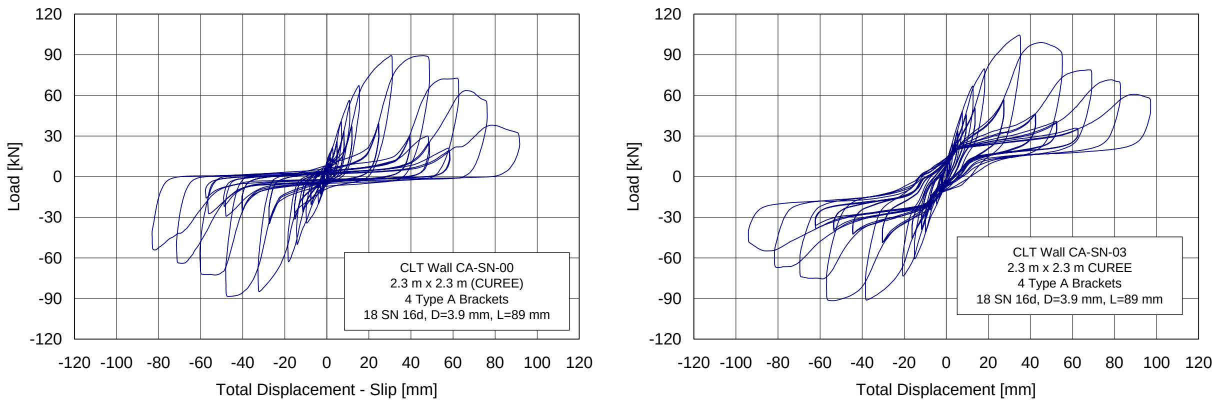


Figure 4. Hysteresis loops for wall 00 with no vertical load and wall 03 with 20kN/m load

Wall 04 with twelve, 10d ring nails per bracket exhibited slightly higher resistance than wall 03 with eighteen 16d spiral nails per bracket. This was mainly due to the higher withdrawal resistance of the ring nails. The ductility of the wall 04, however, was slightly lower than that of wall 03 (Figure 5c). The failure mode observed at the brackets for wall 04 was also slightly different than that of wall 03. While spiral nails in the brackets exhibited mostly bearing failure combined with nail withdrawal, the ring nails in withdrawal tended to pull out small chunks of wood along the way (Fig. 5a and 5b).

The walls with screws in the brackets (05 and 06) reached similar maximum loads as the walls with nails. The load carrying capacity for CLT walls with screws, however, dropped a little bit faster at higher deformation levels than in the case of walls with nails (Figure 6). CLT wall with hold-downs (wall 08A) showed one of the highest stiffness values for a 2.3 m long wall, with its stiffness being 81% higher than that of wall 03 with 18 spiral nails per bracket. CLT wall 08A also showed one of the highest ductility properties (Figure 7b) and therefore this wall configuration is highly recommended for use in regions with high seismicity. Behaviour of one corner of wall 08A during testing is shown in Figure 7a.

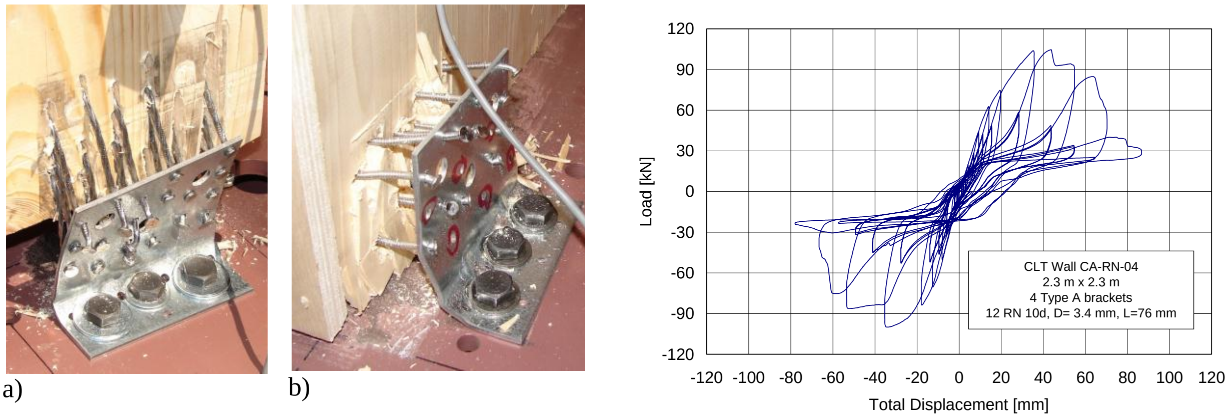


Figure 5. Failure mode of the brackets for: a) wall 02 with spiral nails, and b) wall 04 with ring nails; c) Hysteresis loop for wall 04 with 12 ring nails

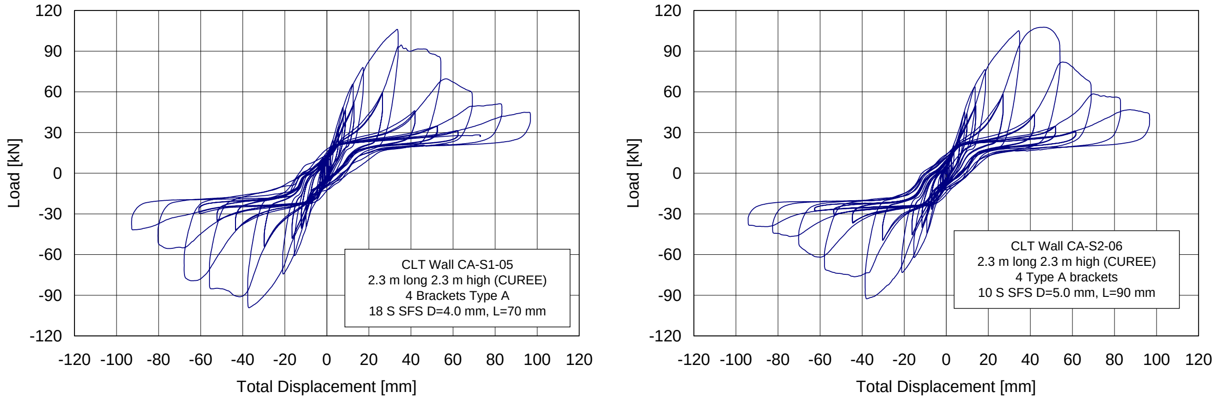


Figure 6. Hysteresis loops: a) wall 05 (18 screws 4x70mm); b)wall 06 (10 screws 5x90mm)

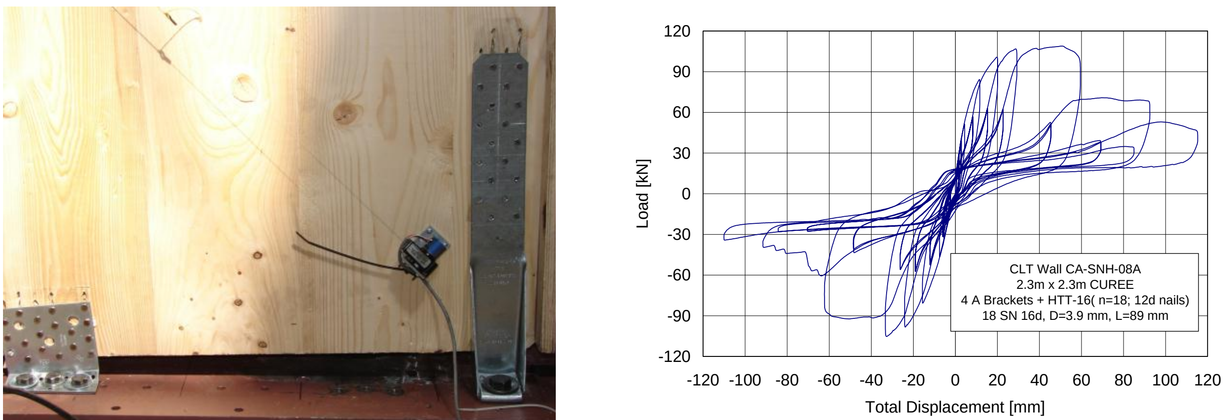


Figure 7. a) A corner of wall 08A during testing; b) Hysteresis loop for the wall 08A

Although timber rivets were developed to be used with glulam, an attempt was made to use rivets in CLT, beside the fact that when driven with their flat side along the grains in the outer layers they will be oriented across the grain in the middle layer. The CLT wall 10A with ten rivets per bracket exhibited by far the highest stiffness than any 2.3 m long wall, with its stiffness being 220% higher than that of wall 03. Timber rivets were also able to carry more load per fastener than any other fastener used in the program. In addition, the wall was able to attain high ductility levels.

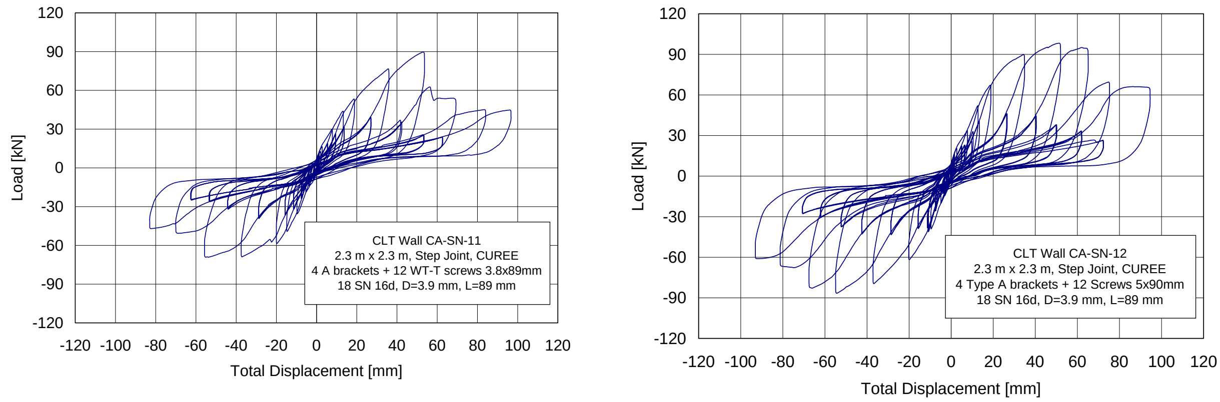


Figure 8. Hysteresis loops for 2-panel walls with different screws the step joint

By introducing a step joint in the wall (creating a wall of two separate panels), the behaviour of the wall was not only influenced by the types of fasteners in the bottom brackets, but also by the type of fasteners used in the step joint. These walls (11 and 12) showed reduced stiffness by 32% and 22% respectively, with respect to the reference wall 03. Both walls were able to shift the occurrence of the yield load F_y and ultimate load F_u at higher deflection levels, while only wall 12 was able to show an increase in its ultimate deflection. Wall 11 that used WT-T screws in the step joint showed reduced ultimate load by 19%, while wall 12 that used regular 5x90mm screws showed a reduction of only 5%. In addition, wall 11 showed higher reduction of ductility compared to the reference wall 03, while the ductility for wall 12 was only slightly lower than that of the reference wall. Based on the results, in case of having multi-panel walls with step joints, the use of regular screws is recommended in high seismic zones. A photo of wall 12 during the testing is shown in Figure 3b, while the behaviour of walls 11 and 12 are shown in Figure 8.

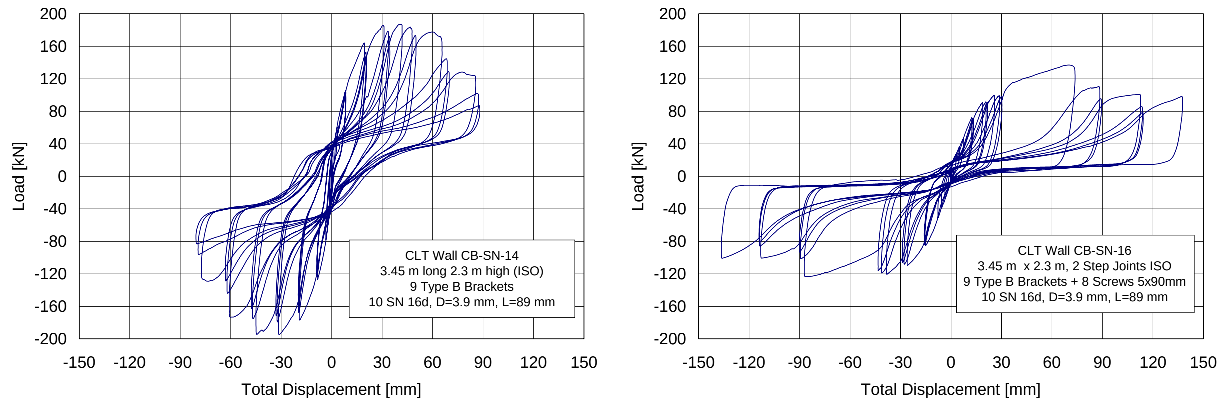


Figure 9. Hysteresis loops for: a) wall 14 (3.45 m long panel); b) wall 16 with 5x90 mm screws in the step joint

The presence of the step joints and the type of fasteners used in them had more significant influence on the overall wall behaviour as the length of the wall increases. Comparing the results from walls 14 and 16, both with length of 3.45 m, a significant change in stiffness and strength properties of the walls was observed. Introduction of step joints enabled wall 16 to carry a significant portion of the maximum load at higher deformation levels (Figure 9b). Specimens 22 and 22B (Figure 10) that were connected to the bottom CLT floor with WT-T type screws placed at 45 degrees showed lower resistance and energy dissipation than any single storey wall in the program (Figure 10b). Grouping the screws at the ends of the panels (wall 22B) created a hold-down effect and helped increase the wall capacity for about 30% compared to that of wall 22. Based on the test results, use of screws at an angle as a primary connector for wall to floor connections is not recommended for structures in earthquake prone regions due to reduced capability for energy dissipation.

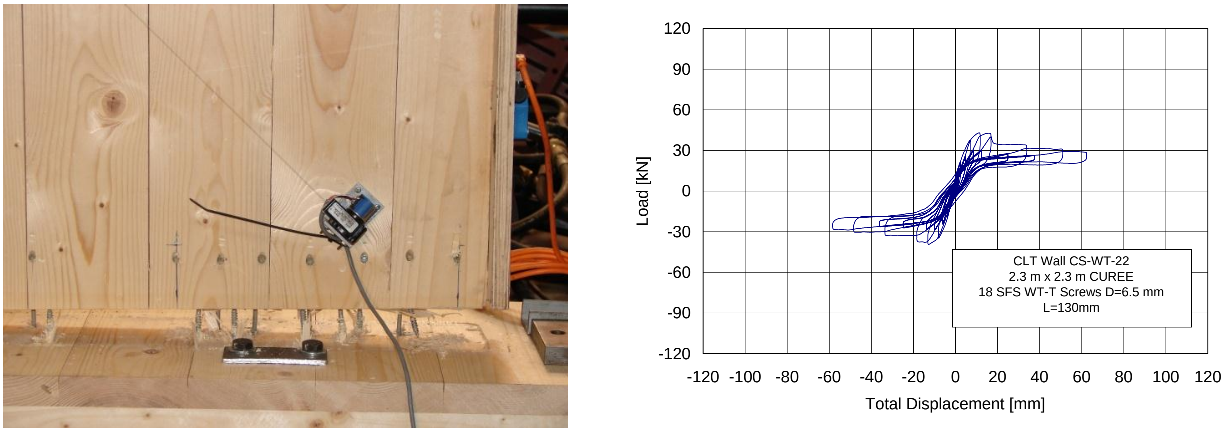


Figure 10. a) A corner of wall 22B during testing; b) Hysteresis loop for wall 22

5. PRELIMINARY ESTIMATES OF FORCE MODIFICATION FACTORS

Force modification factors in building codes (R-factors in Canada, R-factor in the US, and q-factor in Europe) account for the capability of the structure to undergo ductile nonlinear deformations and allow for the structure to be designed for lower seismic forces. In the 2010 NBCC the elastic seismic load is reduced by two R-factors, Ro-factor related to the system over-strength and Rd-factor related to the ductility of the structure. In this section, an estimate will be made for the R-factors for CLT structures based on the equivalency approach in the AC130 acceptance criteria (ICC-ES, 2009). According to the strength limit states of these criteria, assigning an R-factor for a new wood shearwall assembly in the US can be made by showing equivalency of the seismic performance of the new wall assembly in terms of maximum load, ductility, and storey drift obtained from quasi-static cyclic tests, with respect to the properties of lumber-based nailed shearwalls that are already present in the code. Although CLT wall panels as a system differ from wood-frame shearwalls, we can use the AC130 equivalency criteria for preliminary assessment of the R-factors as they are performance-based criteria. The criteria specify that for a new shearwall assembly (in our case CLT) to have the same seismic design factor (R=6.5) as regular shearwalls in the International Building Code (IBC) in the US, the assembly shall satisfy the response criteria given in equations (1) to (3):

$$\frac{\Delta_u}{\Delta_{ASD}} \geq 11 \quad (1); \qquad \Delta_u \geq 0.028 \cdot H \quad (2); \qquad 2.5 \leq \frac{P_{\max}}{P_{ASD}} \leq 5.0 \quad (3);$$

where:

Δ_{ASD} = the displacement at the ASD load level according to the IBC;
P_{ASD} = the assigned Allowable Stress Design load level according to IBC ($P_{max}/2.5$);
Δ_U = ultimate displacement (displacement at which load drops to 80% of the maximum);
H = the height of the panel element;
P_{max} = the maximum load obtained from the backbone curve.

The specified strengths for shearwalls in Canada were soft converted from the ASD values in the US, which were derived as the average maximum load from monotonic tests divided by a safety factor of 2.8, or the average maximum load from cyclic tests divided by 2.5. Here we will assume that the design values for CLT panels have the same safety margin as that of regular wood-frame shearwalls. In addition, as required by the AC130 criteria, only walls tested under the CUREE protocol will be used for the analyses. The average response parameters related to the AC130 criteria obtained from the envelopes of cyclic tests on CLT wall panels are shown in Table 3.

Table 3. The average response parameters for CLT walls as per AC130 criteria

Wall	P_{ASD} [kN]	Δ_{ASD} [mm]	P_{max} [kN]	Δ_u [mm]	Δ_u [% drift]	Ductility Δ_u/Δ_{ASD}
00	35.6	7.8	88.9	66.6	2.9	9.4
02	36.1	8.5	90.3	71.5	3.1	8.5
03	39.2	7.5	98.1	63.6	2.8	8.8
04	40.9	7.5	102.3	59.6	2.6	8.1
05	41.1	8.0	102.7	53.7	2.3	6.8
06	40.0	8.1	100.1	50.1	2.2	6.2
08A	42.8	4.9	107.1	57.8	2.5	13.7
10A	41.0	3.2	102.4	49.0	2.1	16.5
12	37.0	8.6	92.5	72.0	3.1	8.5
14	76.4	3.9	190.9	67.7	2.9	17.7
16	52.1	8.2	130.2	107.1	4.7	13.2
20	60.8	8.6	152.1	70.5	3.1	8.7
21	21.6	3.6	54.1	84.9	3.7	23.8
23	28.9	5.6	72.2	79.8	3.5	15.0
Average for all CLT panels above				68.1	3.0	11.8

Although AC130 criteria do not deal with sets of different walls, one can always look at the average values of the entire set of CLT walls. As shown in Table 3, the average values for the set of CLT walls satisfy the AC130 criteria. The average ductility (as defined in AC130) is 11.8, which is greater than the required minimum of 11 in eq. (1), and the average ultimate storey drift is 3.0%, which is greater than the required 2.8% (eq. 2). Based on this, the CLT walls tested can qualify as new structural wall elements that can share the same seismic response parameters with regular wood-frame shearwalls in the US, which means using an R-factor of 6.5. This value would correspond to having the product of Rd Ro equal to 5.1 in Canada with Rd = 3.0 and Ro = 1.7 being the factors currently used in NBCC for nailed wood-frame shearwalls.

However, at this early stage of acceptance in the design practice, the authors are of the opinion that a bit more conservative set of factors be used for CLT as a structural system. It is recommended that Ro factor of 1.5 and Rd factor in the range of 2.0 to 2.5, to be used as early estimates of R-factors for CLT structures that use brackets with ductile nails or screws and hold-downs. These estimates are in line with the proposed values for the q-factor in Europe (Ceccotti et. al., 2006a, Pozza et. al. 2009). A higher Rd-factor may be considered for CLT systems in the near future, based on the results from additional analytical and experimental work. Based on the research results from tests on braced timber frames, that have already been assigned Ro=1.5 and Rd=2.0 in NBCC (Popovski, 2008), the performance of CLT panels with ductile connections (nails or slender screws) tends to be more ductile. In addition, the CLT as a structural system is far less susceptible to development of soft storey mechanism than braced frames or the other structural systems of the platform type. Since the nonlinear behaviour (and the potential damage) is localized to the bracket and hold-down connection areas only, the CLT panels that are also the vertical load carrying elements, are virtually left intact in place and well connected to the floor panels even after a “near collapse” state is reached. Finally, all walls in one storey in CLT construction contribute to the lateral load resistance, thus providing a system with increased degree of redundancy.

6. CONCLUSIONS

Results from quasi-static tests on CLT wall panels showed that CLT structures can have adequate seismic performance when nails or screws are used with the steel brackets. Use of hold-downs with nails on each end of the wall improves its seismic performance. Use of diagonally placed long screws to connect the CLT walls to the floor below is not recommended in seismic prone areas due to less ductile wall behaviour. Use of step joints in longer walls can be an effective solution not only to reduce the wall stiffness and thus reduce the seismic input load, but also to improve the wall deformation capabilities. Preliminary evaluation of the R-factors for the seismic design of structures according to the NBCC based on the performance comparison to already existing systems in NBCC and on the equivalency performance criteria given in AC130, values in the range of 2.0 to 2.5 for the Rd factor and 1.5 for the Ro factor are suggested as preliminary estimates for CLT structures that use ductile connections such as nails and slender screws.

AKCNOWLEDGEMENT

FPInnovations would like to thank its industry members, Natural Resources Canada (Canadian Forest Service), and the Provinces of British Columbia, Alberta, Saskatchewan, Manitoba, Ontario, Quebec, Nova Scotia, New Brunswick, and Newfoundland and Labrador for their guidance and financial support for this research.

REFERENCES

ASTM E 2126. (2009). Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Load Resisting Systems for Buildings. ASTM International.

Ceccotti A. (2008). New Technologies for Construction of Medium-Rise Buildings in Seismic regions: The XLAM Case. *Structural Engineering International SEI*, **18 (2):**156-165.

Ceccotti A., Follesa M. (2006). Seismic Behaviour of Multi-Storey X-Lam Buildings. *COST E29 Workshop on Earthquake Engineering on Timber Structures*, Coimbra, Portugal.

Ceccotti A., Follesa M., Kawai N., Lauriola M.P., Minowa C., Sandhaas C., Yasumura M. (2006a). Which Seismic Behaviour Factor for Multi-Storey Buildings made of Cross-Laminated Wooden Panels? *Proceedings 39th CIB W18 Meeting*, **39-15-4**, Firenze.

Ceccotti A., Follesa M., Lauriola M.P., Sandhaas C. (2006b). Sofie Project – Test Results on the Lateral Resistance of Cross-Laminated Wooden Panels. *First European Conference on Earthquake Engineering and Seismicity*, Geneva, Switzerland.

Dujic B., Strus K., Zarnic R., Ceccotti A. (2010). Prediction Of Dynamic Response of a 7-Storey Massive Xlam Wooden Building Tested On a Shaking Table. *11th World Conference on Timber Engineering*, Riva del Garda, Italy.

Dujic B., Klobcar S., Zarnic, R. (2007). Influence of Openings on Shear Capacity of Wooden Walls. *Proceedings of the 40th CIB-W18 Meeting*, **40-15-6**, Bled, Slovenia.

Dujic B., Hristovski V., Stojmanovska M., Zarnic R. (2006). Experimental Investigation of Massive Wooden Wall Panel Systems Subjected to Seismic Excitation. *First European Conference on Earthquake Engineering and Seismicity*. Geneva, Switzerland.

Dujic B., Pucelj J., Zarnic R. (2004). Testing of Racking Behavior of Massive Wooden Wall Panels. *Proceedings of the 37th CIB-W18 Meeting*, **37-15-2**, Edinburgh, Scotland.

Fragiacomo M., Dujic, B., Sustersic, I. (2011). Elastic and ductile design of multi-storey crosslam massive wooden buildings under seismic actions. *Engineering Structures*. Special Issue - Modelling the Performance of Timber Structures. **33:11**, 3043-3053.

Fragiacomo, M., and Rinaldin, G. (2011). Advanced models for seismic analyses of timber buildings. Außergewöhnliche Einwirkung - Erdbeben - im Holzbau. Workshop on timber construction. Grazer Holzbau-Fachtagung–9. Gra FHT'11. Technical University of Graz, Austria.

Gavric I., Ceccotti A., Fragiacomo M. (2011). Experimental cyclic tests on cross-laminated timber panels and typical connections. Außergewöhnliche Einwirkung - Erdbeben - im Holzbau. Workshop on timber construction. Grazer Holzbau-Fachtagung–9. Gra FHT'11. Technical University of Graz, Austria.

ICC-ES AC130. (2009). Acceptance Criteria for Prefabricated Wood Shear Panels. ICC Evaluation Services, Inc. Whittier, California.

Popovski, M., Karacabeyli, E. (2008). Force Modification Factors and Capacity design Procedures for Braced Timber Frames. *Proceedings 14th WCEE*, Beijing, China.

Popovski, M., Karacabeyli, E., Ceccotti, A. (2011). Seismic Performance of Cross-Laminated Timber Buildings. Chapter 4 of the FPInnovations' CLT Design Handbook, Canadian Edition.

Pozza, L., Scotta, R., Vitaliani, R. (2009). A non linear numerical model for the assessment of the seismic behaviour and ductility factor of X-lam timber structures. *International Symposium on Timber Structures*. Istanbul, Turkey. 151-162.

Schädle, P., Blaß, H.J. 2010. Earthquake behaviour of modern timber construction systems. *Proceedings of the 11th World Conference on Timber Engineering*, Riva del Garda, Italy.