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PERFORMANCE OF CROSS LAMINATED TIMBER SHEAR WALLS UNDER CYCLIC LOADING

Shahnewaz, Md[1,5], Tannert, Thomas[2], Alam, M. Shahria[3] and Popovski, Marjan[4]
[1] The University of British Columbia, Vancouver, Canada
[2] University of Northern British Columbia, Prince George, Canada
[3] The University of British Columbia, Kelowna, Canada
[4] FPInnovations, Vancouver, Canada
[5] md.shahnewaz@alumni.ubc.ca (corresponding author)

Abstract: Cross-laminated timber (CLT) is becoming increasingly popular as a structural material for residential and non-residential buildings because of its low carbon foot-print and potential cost-competitiveness compared to concrete and steel. The 2016 supplement to the 2014 edition of the Canadian Standard for Engineering Design in Wood, CSAO86, provides provisions for platform type CLT structures. The research presented in this paper investigated the seismic behavior of CLT shear walls with different connections for platform-framed construction. Finite element analyses were conducted in OpenSees with various connections including steel brackets and hold-downs. The CLT panels were modelled as orthotropic elastic material, and non-linear springs were used for the connections. The models for the hysteretic behaviour of the connections under cyclic loading and the wall assemblies were calibrated using experimental tests. A parametric study was conducted that evaluated the wall capacity and stiffness as a function of number and type of connectors. It was shown that increasing the number of the connectors from 4 to 7, the average capacity of CLT shear walls increased from 53-57% while the stiffness of the CLT shear walls increased by 33-39%.

1 INTRODUCTION

Cross-laminated timber (CLT) is an engineered wood product categorized as “mass” timber. The use of CLT is increasingly gaining popularity because of its many benefits when compared to either light-fame wood or concrete and steel construction. CLT buildings have a low carbon footprint due to low embodied energy, low greenhouse gas emissions and the carbon storage capacity of wood. The good thermal insulation and a fairly good behaviour in case of fire are added benefits. Furthermore, it is a clean product to work with resulting in less waste and dust produced on site which is better in terms of health and safety (Gagnon and Pirvu 2012). CLT panels consist of several layers, usually an uneven number, of lumber boards stacked crosswise and glued together. Pre-cut wall and floor panels are assembled on the construction site using various types of screws and steel connectors to form the structural system.

Until today, platform-type construction, where each floor is acting as a platform for next floor, is the most common structural system used for low to mid-rise CLT buildings. The building is either directly connected to a reinforced concrete (RC) foundation or on top of an RC podium. The walls are connected to foundations or RC podium by steel brackets and hold-downs using metal fasteners like screws and nails. The CLT walls to the floors in the upper storeys are connected by brackets and/ hold-downs or by self-tapping screws (STSs). The panels in floors and walls are connected using either lap or spline joints.

The CLT shear walls act as the lateral load resisting system in a CLT platform building. It is important to understand the actual behaviour of CLT shear walls under lateral loading. Recent research focused to predict the behavior of CLT walls under cyclic loading. Popovski et al. (2010) performed quasi-static monotonic tests on 32 CLT shear walls with hold-downs and bracket connections using various fasteners, i.e., ring nails, spiral nails, STSs, and timber rivets. Full-scale tests on CLT walls were conducted at FPInnovations, Vancouver, Canada. The panels were 2.3m x 2.3m made of 3-ply (94mm thick) European spruce. The test setup for CLT shear wall is shown in Figure 1a. Steel brackets and hold-downs connectors with various fasteners (annular ring nails, spiral nails, screws, and timber rivets) were used for the wall-to-foundation connections (Figures 1b and 1c). Their results showed that the nonlinear behaviour of the walls was localized at the connections only.

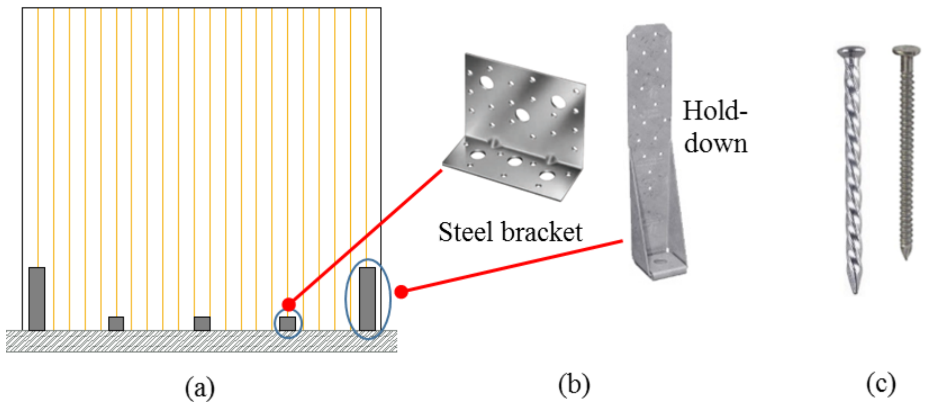

Figure 1: (a) Schematic of CLT shear wall, (b) steel bracket and hold-down, and (c) fasteners

Gavric et al. (2015a) performed cyclic tests CLT walls with angle bracket and hold-downs (Figure 1b) using annular ring nails fasteners (Figure 1c) for wall-to-foundation connections. The 5-ply panels were 3m x 3m with a thickness of 85mm. Brackets and hold-downs with annular ring nails were used for wall-to-foundation connections. They observed similar behavior that the failure of the systems was located mostly at the connections, while the CLT wall panels were subjected to negligible in-plane deformations. In addition to experimental investigations, the behavior of CLT walls under monotonic and cyclic loading was also studied using finite element analysis (FEA). Shahnewaz et al. (2017a) conducted FEA on CLT walls with openings. Based on a parametric study, they proposed simplified equations for calculating the in-plane stiffness of CLT walls with openings. Furthermore, Shahnewaz et al. (2017b) performed sensitivity analyses which identified the important parameters that affect the in-plane stiffness of CLT walls, namely the ratio of opening to wall area and the aspect ratio of opening to wall.

The connections in CLT shear walls experience all the non-linear deformation. Therefore, some researchers focused on to predict the actual behaviour of various CLT connectors in a shear wall system. Gavric et al. (2015b) conducted monotonic and cyclic tests on two different types of hold-downs and bracket connections with annular nails. They tested both CLT wall-to-foundation and wall-to-floor connections using 5-ply of 85 mm CLT wall panels and 5-ply of 142 mm of CLT floor panels and observed that brackets have similar capacity and stiffness in under tension and shear tests. On the contrary, hold-downs showed higher strength and stiffness in tension compared to bracket connections and their sliding resistance was negligible. A similar study confirmed these findings on bracket connections using three different types of fasteners i.e., spiral nails, ring nails, and STS (Schneider et al. 2015).

However, little is known about the seismic behaviour of CLT shear walls as a function of the number and type of connectors which is important for a reliable seismic design. Therefore, the present paper investigated the cyclic behavior of CLT shear walls in platform-type buildings.

2 FINITE ELEMENT ANALYSIS

2.1 FEA Model of CLT Shear Wall

FEA models of CLT shear walls were developed in OpenSees (McKenna et al. 2000). Test results on CLT shear walls showed that CLT panel acted as a rigid body and all the non-linear deformation occurred at connections (Popovski et al. 2010). To represent the actual kinematic behaviour of CLT shear walls, the CLT panels were modelled using plane-stress shell element (shown in Figure 2 left) with elastic material properties. The metal connectors were modelled using non-linear zero-length springs with "pinching4" hysteresis properties as shown in Figure 3. Both Gavric's and Schneider's tests (Gavric et al. 2015a and Schneider et al. 2015) showed that bracket connections have similar resistance to cyclic shear and tension, therefore, each bracket connectors were modelled with two-orthogonal zero-length springs at the same location. The orthogonal zero-length springs simulate the sliding and uplift of the shear walls. On the contrary, the hold-downs were modelled using only a single zero-length spring to resist uplifting by neglecting their shear resistance as observed in hold-downs tests (Gavric et al. 2015a).

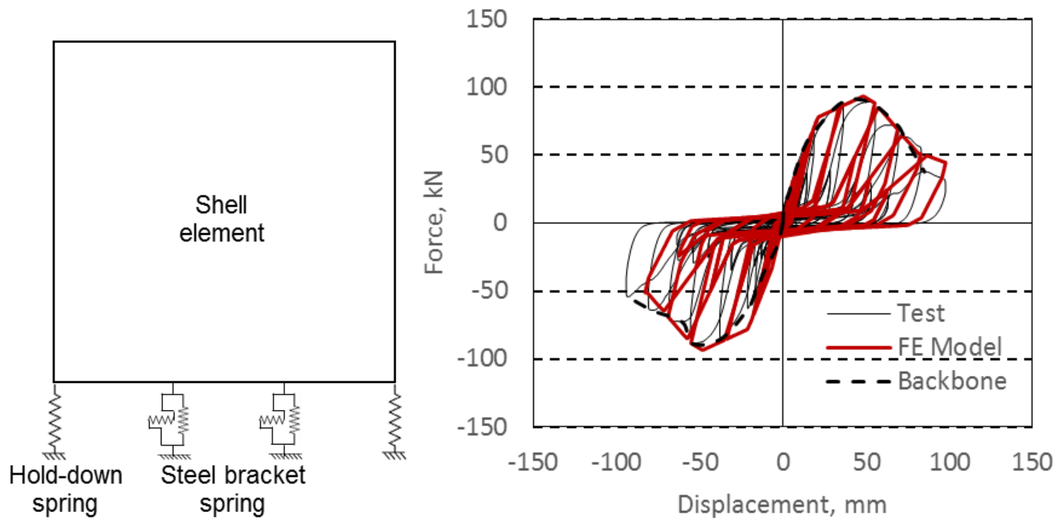

Figure 2: FEA model for CLT shear wall and (left) FEA vs test: load-deformation curves (right)

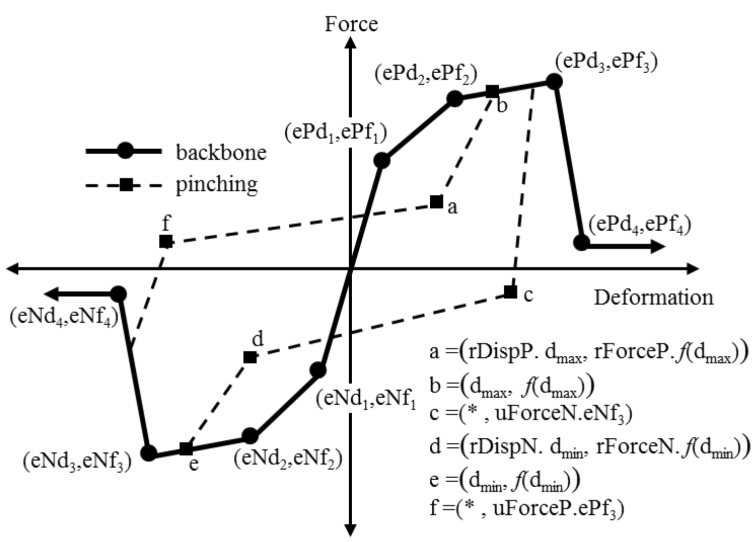

Figure 3: Pinching 4 model

2.2 Calibration of CLT Connections

The properties of the CLT connections used in the FEA for different types of connectors were calibrated from test results (Gavric et al. 2015b and Schneider et al. 2015). Gavric's and Schneider's connections were tested using 85mm and 94mm CLT panels, respectively. A wall configuration with steel brackets and hold-downs was shown in Figure 1. Two types of Simpson Strong-Tie steel brackets (Bracket A: 90mm×48mm×116mm and Bracket B: 90mm×105mm×105mm) with different fasteners were tested. Gavric et al. (2015b) also performed tests on two types of Simpson Strong-Tie hold-downs e.g., HTT16 and HTT22 using ring nails fasteners. The connections are designated as B1 to B5 for brackets and HD1 to HD2 for hold-downs throughout this article as shown in Table 1.

Table 1: CLT connection Types

Connection Type	Connection ID	Fasteners
Bracket A: 90mm×48mm×116mm	B_1	18-16d SN 3.9×89 mm
	B_2	18-SFS screw 4×70 mm
	B_3	10-SFS screw 5×90 mm
	B_4	12- RN 3.4×76 mm
	B_5	11- RN 4×60 mm
Hold-down: HTT22	HD_1	12- RN 4×60 mm
Hold-down: HTT16	HD_2	18-16d SN 3.9×89 mm

FEA models of CLT connections were also developed in OpenSees (McKenna et al. 2000) to validate the metal and shear connectors from Gavric et al. (2015b) and Schneider et al. (2015). Both tension and shear connections were modelled using zero-length spring element. It is defined by two nodes at the same location where the nodes are connected by uniaxial material model “Pinching4” to represent the force-deformation relationship for the element. The least-square method was employed to estimate the parameters of the Pinching4 model from the backbone of the tested connections (Figure 4).

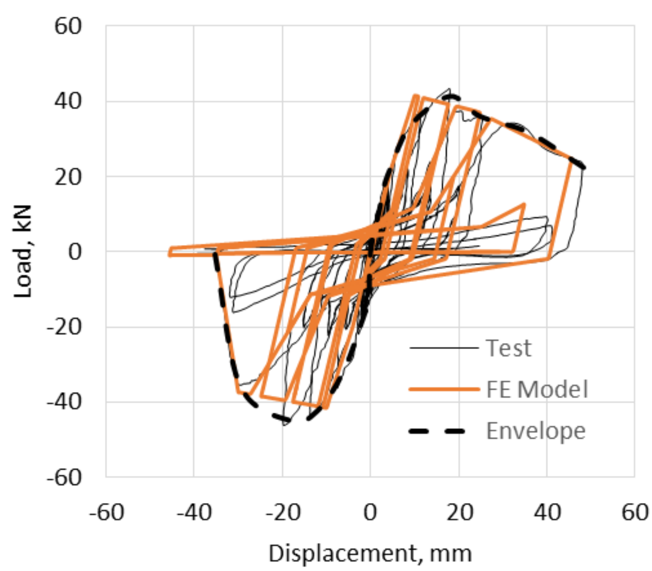

Figure 4: FEA vs test: load-deformation of bracket connection under cyclic shear

2.3 FEA vs. test results on CLT shear walls

Eight CLT shear walls from the FPInnovations tests (Popovski et al. 2010) and two shear walls from Gavric et al. (2015a) tests were validated (Table 2). The hysteresis loops along with their backbone curves indicate a good agreement in between tests and FEA (Figure 2 right). From the backbone curves, the important parameters representing the seismic performance of the CLT shear walls such as stiffness, strength, and ductility were calculated. All parameters were computed based on equivalent energy elastic plastic (EEEP) curve (ASTM E2126 2011).

Table 2: FEA vs test results for CLT shear walls

Test Wall ID	No. of brackets (Type)	No. of hold-downs (Type)	Vertical Load	P_{peak} (kN)			E (kN-m)			D	K_e
			kN/m	FE	test	%Δ	FE	test	%Δ	-	kN/mm
[1]CA-SN-00	4 (B_1)	-	0.0	93.3	88.9	5.0	26.4	27.8	5.1	3.5	4.4
[1]CA-SN-02	4 (B_1)	-	10.0	96.4	90.3	6.8	28.8	30.5	5.8	3.9	4.7
[1]CA-SN-03	4 (B_1)	-	20.0	99.6	98.1	1.5	29.9	31.0	3.5	4.5	4.9
[1]CA-S1-05	4 (B_2)	-	20.0	97.8	102.7	4.8	25.6	28.1	8.9	3.3	4.9
[1]CA-S2-06	4 (B_3)	-	20.0	92.9	100.1	7.2	25.0	26.9	7.1	3.3	4.6
[1]CA-RN-04	4 (B_4)	-	20.0	99.3	102.3	2.9	25.6	26.8	4.8	3.4	5.4
[1]CA-SN-20	7 (B_1)	-	20.0	153.9	152.1	1.2	44.3	45.5	2.7	4.2	7.2
[1]CA-SNH-08	3 (B_1)	2 (HD_2)	20.0	126.2	118.2	6.8	36.4	37.7	3.3	3.9	7.3
[2]I.1	2 (B_5)	2 (HD_1)	18.5	75.0	70.7	6.1	12.6	13.1	4.4	3.0	5.0
[2]I.2	4 (B_5)	2 (HD_1)	18.5	106.7	104.2	2.4	21.9	24.1	9.1	3.5	5.9

Note: [1]Popovski et al. (2010), [2]Gavric et al. (2015a)

3 PARAMETRIC STUDY

A parametric study was performed on single CLT shear walls with the variation of the number and types of brackets and hold-downs. Two types of shear walls were considered: Case A: CLT shear wall with brackets only and Case B: CLT shear walls with brackets and hold-downs. The CLT panels were 2.3×2.3m with 3-ply of 94mm thick. The shear walls with brackets were analyzed with five different types of fasteners (B1 to B5 in Table 1). The number of brackets was changed from 4 to 7. In case of CLT shear walls connected by brackets and hold-downs (Case B), two types of hold-downs (HD1 or HD2) were considered at each corner of the panel. The number of brackets was varied from 2 to 5, therefore, the total number of connectors remain same as Case A (CLT shear walls with brackets only). The shear walls were analyzed under CUREE loading protocol and the parameters of the shear wall stiffness, strength, and ductility were calculated from the resulting EEEP curves.

The single CLT shear wall's capacity and stiffness as a function of the type and number of connectors are plotted in Figures 5 and 6, respectively. Both, the capacity and stiffness, increase with the number of connectors. By increasing the number of the connector from 4 to 7, the average capacity and stiffness in CLT shear walls with brackets increases by 57% and 39%, respectively (Figure 5). Similarly, the average capacity and stiffness increase in the CLT shear walls with brackets and hold-downs was found 53% and 33%, respectively (Figure 6).

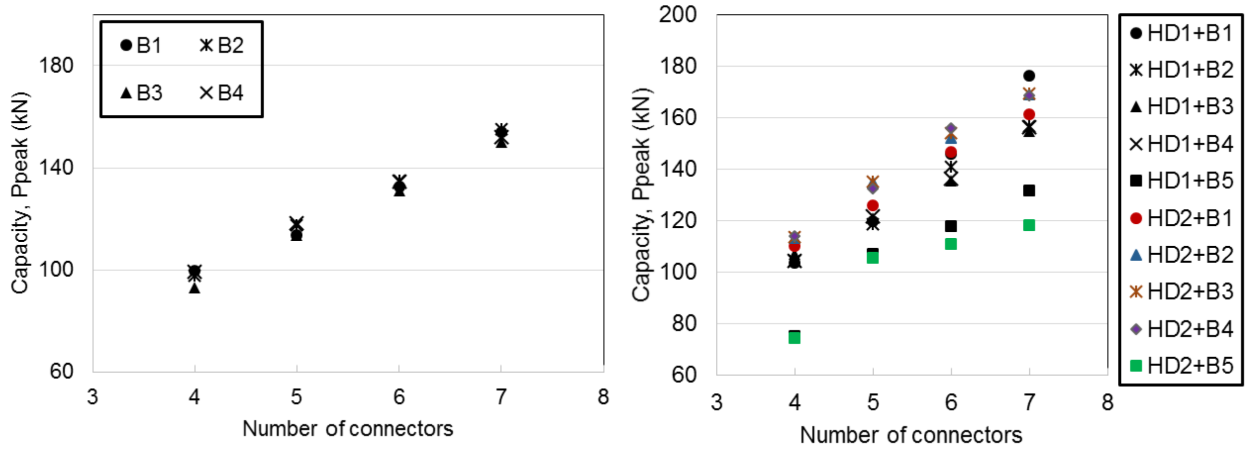

Figure 5: Capacity of CLT shear walls: with brackets only (left), with brackets and hold-downs (right)

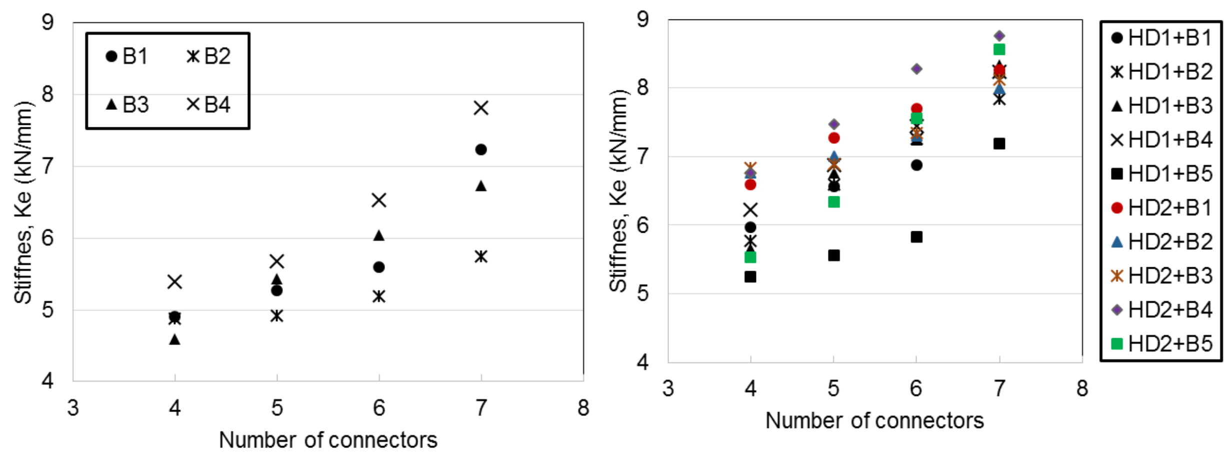

Figure 6: Stiffness of CLT shear walls: with brackets only (left), with brackets and hold-downs (right)

4 CONCLUSION

The research presented in this paper studied the behaviour of CLT shear walls under cyclic loading. FEA models were developed in OpenSees and validated against full-scale shear wall tests conducted by Popovski et al. (2010) and Gavric et al. (2015a). Steel brackets and hold-downs were considered as connectors with various types of fasteners. The CLT connections were modelled by using "pinching4" hysteresis in OpenSees where the model parameters were calibrated against connection tests conducted by Gavric et al. (2015b) and Schneider et al. (2015). Finally, the performance of CLT shear walls was evaluated by conducting a parametric study. The capacity and stiffness of the shear walls were calculated with the variation of number and types of connectors. It was shown that increasing the number of the connectors from 4 to 7, the average capacity and stiffness in CLT shear walls increased by 53-57% and 33-39%, respectively.

Acknowledgements

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