Lateral Performance of Cross-laminated Timber Shear Walls: Analytical and Numerical Investigations

Md Shahnewaz[1*], Thomas Tannert[2], and Marjan Popovski[3]

[1*] *Postdoctoral Research Fellow, University of Northern British Columbia, Canada*
[2] *Associate Professor, Wood Engineering, University of Northern British Columbia, Canada*
[3] *Principal Scientist, FPInnovations, Vancouver, Canada*
[*] *Corresponding author's e-mail: md.shahnewaz@unbc.ca*

ABSTRACT

Cross-laminated timber (CLT) is becoming a viable option for mid-rise buildings in North America. CLT walls are very effective in resisting lateral forces resulting from wind and seismic loads, yet no standard provisions are available to estimate the resistance of CLT shear walls under lateral loading. The present research investigated CLT shear wall's performance by evaluating the preferred kinematic rocking behaviour. An analytical procedure was proposed to estimate the resistance of CLT shear walls in a platform type construction. Finite element models of CLT shear with various brackets and hold-downs connections were developed. The models were validated against experimental results. Furthermore, a parametric study on CLT shear walls with the variation of type and number of connectors was conducted. The resistance estimated from parametric study and against analytical were compared. The proposed formulas can be useful tool for the design of CLT platform-type buildings, however, require further experimental validation.

KEYWORDS

Timber building; Cross-laminated Timber; Shear Wall; Earthquake; design code; finite element.

INTRODUCTION

With an increase in sustainable construction, cross-laminated timber (CLT) is becoming a popular green material for timber buildings due to its good thermal insulation, adequate fire resistance, speedy construction, and is effective in resisting lateral forces (Gagnon and Pirvu, 2011). CLT panels are usually three to nine layers of boards glued orthogonally to each other to form a solid panel which can be used for wall, floor and roof assemblies. CLT shear walls can be used as the main lateral force resisting system for platform-type construction. CLT shear walls are connected to the floor diaphragms by hold-downs (HDs) and brackets. The connects and the CLT panels are the two main components of CLT shear walls in a platform-type construction. The CLT shear walls are connected to the foundation, the concrete podium, or the CLT slab below by steel brackets and HDs using metal fasteners such as screws or nails. CLT walls are connected to the CLT slabs above either using long self-tapping screws (STS) or brackets. The individual wall panels in coupled walls are most often connected by screws using either lap or spline joints.

CLT has created an opportunity to build mid to high rise timber building for its high in-plane stiffness and strength properties (Shahnewaz et al., 2018) and with the innovation of ductile connections (Zhang et al., 2018). The in-plane bending and shear stiffness of CLT panels was investigated by Brandner et al. (2017). Shahnewaz et al. (2016, 2017) developed finite element analyses (FEA) models of CLT panels with openings and proposed analytical expression to estimate the in-plane stiffness of CLT walls with the consideration of size, shape and location of openings. Additionally, a sensitivity analysis was performed to identify the important parameters affecting the in-plane stiffness of CLT panels with openings (Shahnewaz et al., 2017). They also investigated the stiffness, strength and resistance of CLT walls with various connections and found that walls with brackets and HDs exhibited better strength and stiffness performance compared to walls without HDs (Shahnewaz et al., 2018, 2019). Gavric et al. (2015a, 2015b) conducted monotonic and cyclic tests on different hold-downs, brackets and shear connections for wall-to-foundation, -floor, -wall and -floor connections. They observed that brackets have similar capacity and stiffness under tension and shear loading. Similar study by Schneider et al. (2015) on bracket connections using three types of fasteners: spiral nails, ring nails and STS also confirmed the findings from Gavric et al (2015a, 2015b).

The Canadian standard for engineering design in wood (CSA-O86 2016) includes comprehensive design provisions for CLT shear walls. These provisions apply to platform-type construction not exceeding 30 m in height, respectively 20 m for high seismic zones. It is important to understand and quantify the actual behaviour of CLT walls under lateral loads for a reliable design of CLT buildings. Therefore, the objective of this study is to evaluate and quantify the lateral performance of CLT shear walls with various types of connections for platform-type construction.

LATERAL RESISTANCE OF CLT SHEAR WALLS

Under lateral loading, CLT shear walls can experience three types of kinematic behavior i.e., sliding (Figure 1a), rocking (Figure 1b) and a combination of sliding and rocking of the wall (Figure 1c). The new provisions of CSA-O86 limit the permitted kinematic motion to rocking only, therefore only rocking is discussed in the following section.

CLT shear wall with brackets

The rocking behaviour of CLT shear wall is shown in Figure 2. It has been assumed that the CLT wall rotates about the right corner point under lateral loading, therefore, the reactions of the connectors follow a triangular distribution. The rocking resistance of the CLT shear wall can be calculated by taking summation of the moment at the lower right corner of the wall:

$$F_r\ h = N_{1y}x_1 + N_{2y}x_2 + N_{3y}x_3 + N_{4y}x_4 + q\frac{b^2}{2} \tag{1}$$

where N_{iy} is the rocking reaction of each connection, x_i is the distance of each connector from the right corner, b is the width of the CLT panel, q is the vertical load on top of the panel. The rocking resistance of the CLT shear wall is reached when the first bracket (left corner) has reached its ultimate resistance -i.e. $N_{1y} = N_B$. As seen in Eq. (1), the first bracket carries the maximum moment due to the rocking of the wall since it has the highest lever arm –i.e. it locates at a maximum distance from the right side of the wall, x_1. Therefore, when it reaches its ultimate resistance (i.e. fails) the rocking resistance of the wall will reduce immediately with the subsequent failure of the rest of the brackets. Therefore, using the triangular distribution of the bracket's forces in Figure 2a, the reaction forces of the brackets can be written as:

$$N_{1y} = N_B;\ N_{2y} = (x_2 / x_1)N_B;\ N_{3y} = (x_3 / x_1)N_B;\ N_{4y} = (x_4 / x_1)N_B \tag{2}$$

Substituting Eq. (2) into Eq. (1), the rocking resistance of the wall can be estimated as:

$$F_r = \frac{N_B}{h x_1}\left(\sum_{1}^{n_B} x_i^2\right) + q\frac{b^2}{2h} \quad (3)$$

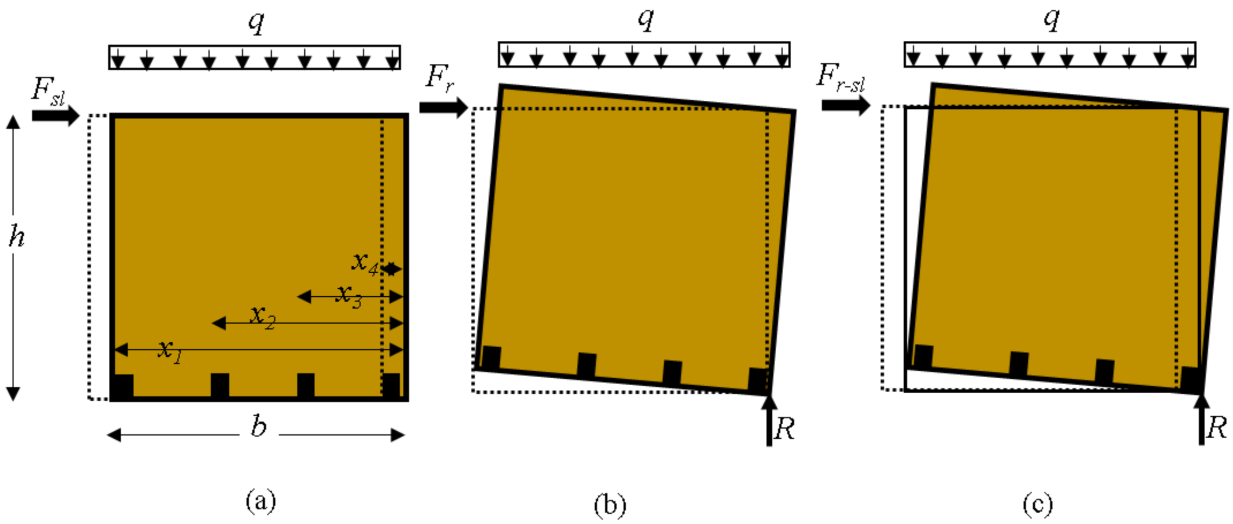

Figure 1. CLT wall kinematic motions: (a) sliding, (b) rocking and (c) combined sliding-rocking

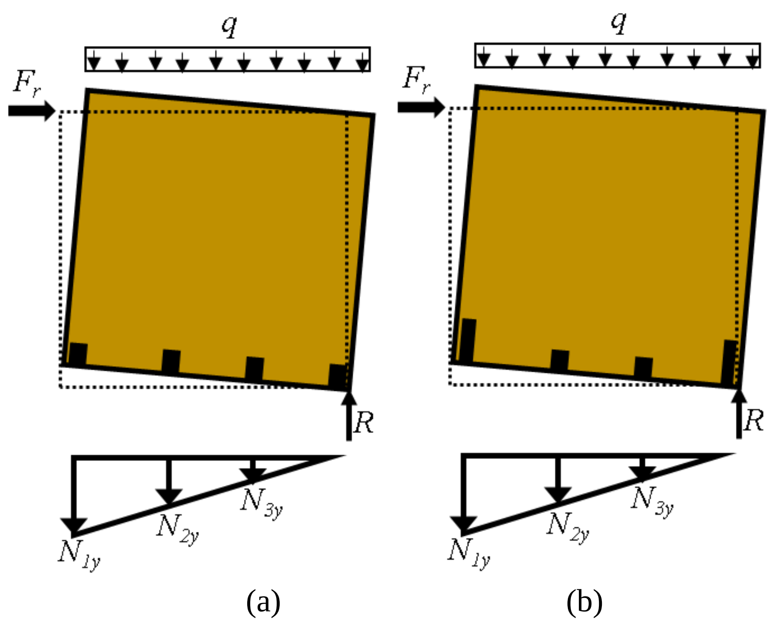

Figure 2. Rocking of CLT walls: (a) with brackets only and (b) with brackets and HDs

CLT shear wall with brackets and HDs

The rocking resistance of the CLT shear wall's hold-down and bracket connections can be calculated by taking a summation of the moment at the lower right corner of the CLT shear wall (Figure 2b):

$$F_r\, h = N_{1y}x_1 + N_{2y}x_2 + N_{3y}x_3 + N_{4y}x_4 + q\frac{b^2}{2} \tag{4}$$

The rocking resistance of the CLT shear wall will be reached when the left hold-down has reached its ultimate resistance -i.e. $N_{1y} = N_{HD}$. The rocking reaction for the remainder of the brackets will follow the triangular load distribution as shown in Figure 2b. The resistance of the two intermediate brackets can be calculated following the triangular distribution as:

$$N_{1y} = N_{HD};\, N_{2y} = (x_2 / x_1)N_B;\, N_{3y} = (x_3 / x_1)N_B;\, N_{4y} = (x_4 / x_1)N_{HD} \tag{5}$$

Substituting Eq. (5) into Eq. (4), the resistance of the wall under rocking can be written as:

$$F_r = \frac{N_{HD}x_1}{h} + \frac{N_B}{x_1 h}\left(\sum^{n_B} x_i^2\right) + q\frac{b^2}{2h} \tag{6}$$

FINITE ELEMENT ANALYSIS

Finite Element Analyses (FEA) models of CLT shear walls were developed in OpenSees (2000). The FEA models of CLT walls were developed based on experimental research conducted at FPInnovations, Vancouver Canada (Popovski et al. 2010). The walls were made of 3-ply (94 mm thick) European spruce panels. 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. The CLT panels were modelled using plane-stress shell elements with elastic material properties and the metal connectors were modelled using non-linear zero-length springs with the "Pinching4" hysteresis model.

A parametric study was performed on 56 CLT shear walls with variations in the number and types of brackets and hold-downs. Two types of shear walls were considered: a) CLT shear wall with brackets only, and b) CLT shear walls with both brackets and HDs. The CLT panels were 2.3 m × 2.3 m 3-ply and 94 mm thick. The shear walls with brackets were analyzed with five different types of fasteners (B_1 to B_5, see Table 1), with the number of brackets varying from 4 to 7. Two types of hold-downs (HD_1 or HD_2) were considered for the case of CLT walls with brackets and HDs. Reverse cyclic displacements were applied in the top nodes of the FEA wall models following CUREE loading protocol.

The resistances of the 56 CLT shear walls, F_d were calculated using the proposed formulas. The ratio of the peak load to calculated resistance (P_{peak}/F_d) -i.e. a factor of safety was estimated as seen in Figure 3 and Figure 4. The figures clearly illustrate that there is no safety margin when considering sliding –i.e. the calculated sliding resistance and the peak loads from the FEA were very close. By contrast, the rocking and the combined rocking-sliding kinematic motion produced conservative results. Additionally, shear walls with HDs provided a higher safety margin when compared to the shear walls with brackets only. The average P_{peak}/F_d for shear walls with brackets and HDs was found to be higher (i.e. rocking = 2.3 and combined rocking-sliding = 2.7) when compared to the average capacity of walls with brackets only (i.e. rocking = 1.9 and combined rocking-sliding = 2.2).

Table 1. Connections for CLT Walls

Connection Type		ID	Fasteners
Brackets	90×48×116 mm	B1	18 16d SN 3.9×89 mm
		B2	18 SFS screw 4×70 mm
		B3	10 SFS screw 5×90 mm
		B4	12 RN 3.8×76 mm
		B5	11 RN 4×60 mm
Hold-downs	HTT16	HD1	9 RN 4×60 mm
	HTT22	HD2	12 RN 4×60 mm

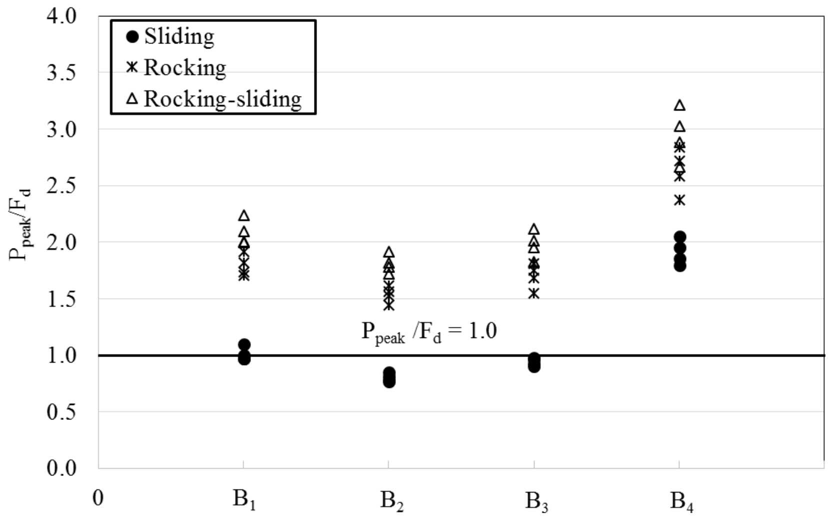

Figure 3. Peak vs resistance of CLT shear wall with brackets only.

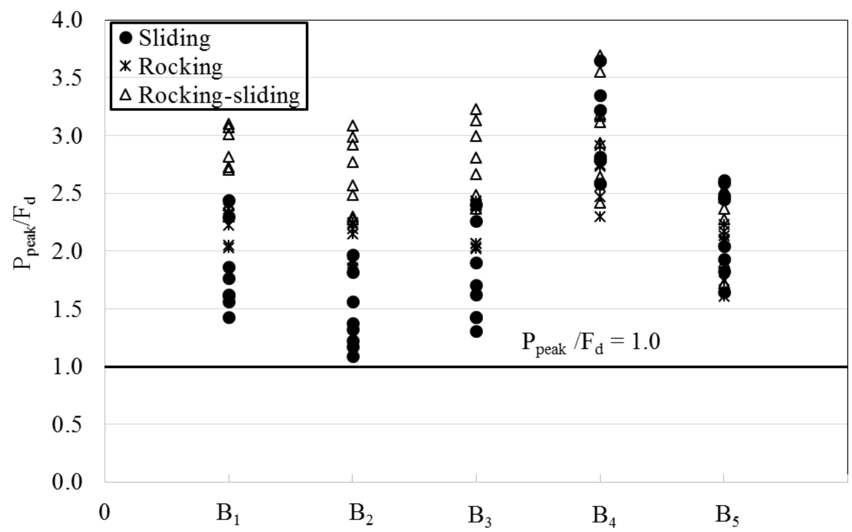

Figure 4. Peak vs resistance of CLT shear wall with brackets and HDs.

CONCLUSION

This study presented a procedure to estimate the performance of CLT shear walls under lateral loading. The kinematic behaviours of CLT shear walls due to sliding, rocking and a combination of rocking and skidding were evaluated. Formulas to estimate the lateral resistance of CLT shear walls were proposed for the rocking motion which is the only one permitted in CSA-O86. FEA models of CLT walls were developed and validated from experiment. A parametric study was conducted with variation on the number and types of connectors. The estimated resistances using the proposed formulas were compared against FEA results.

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