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EXPERIMENTAL VALIDATION OF PROPOSED CAPACITY-BASED DESIGN APPROACHES FOR MULTI-PANEL CLT SHEARWALLS

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ABSTRACT: Several research studies have proposed capacity-based design (CD) approaches for multi-story buildings containing cross-laminated timber (CLT) shearwalls. The current study contributes to the state of knowledge by evaluating experimental tests and numerical models on multi-panel CLT shearwalls in context of a proposed CD method available in the literature. The evaluation includes the yield hierarchy among energy dissipative elements and requirements to ensure sufficient energy dissipation is achieved in the shearwall. The results from the experimental tests on conventional connections used in CLT shearwalls are presented, as they are used as inputs in the numerical model and CD expressions. Reasonable correlation was found between the results obtained from experimental tests and numerical models. The walls' shear resistance obtained from the CD expressions represented a load level lower than the maximum resistance obtained from the experimental tests and numerical modes. This additional strength could be attributed to the contribution of angle brackets after the yielding of the hold-down, which is represented in the tests and numerical models but not in the CD equations. It was demonstrated that satisfying the proposed equations would lead to the desired lateral behaviour and energy dissipation in CLT shearwalls.

KEYWORDS: Cross-Laminated Timber (CLT); Capacity-based Design (CD); experimental tests; connections; seismic load

1 INTRODUCTION

The performance of cross-laminated timber (CLT) shearwalls subjected to seismic loads is mainly dominated by behaviour of the mechanical connections, which provide resistance and energy dissipation in the system, while the CLT panels can be reasonably assumed to behave like rigid bodies. Experimental tests have demonstrated that multi-panel CLT shearwalls possess better seismic performance than shearwalls consisting of a single panel, since joints connecting the panels together provide significant ductility (e.g., [1], [2]). This is also emphasized in the current version of the Canadian Engineering Design in Wood standard, CSA O86-19 [3], where panel-to-panel connections are defined as one of the main energy-dissipative elements in CLT shearwalls. A typical multi-panel CLT shearwall consists of panels attached together using panel-to-panel connections, and mechanical anchors, such as hold-down and angle brackets, which connect the wall to the foundation or floors, as shown in Figure 1.

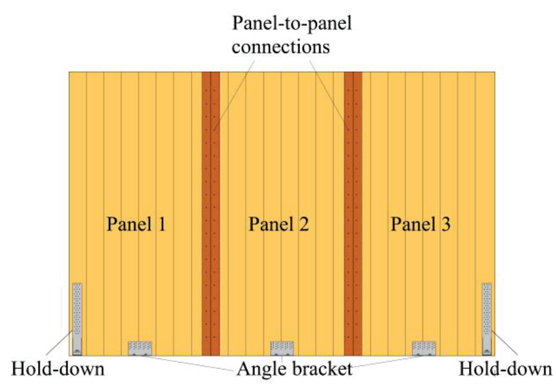

Figure 1: *Example of a multi-panel CLT shearwall*

Recent research has developed and proposed capacity-based design (CD) approaches for multi-story buildings containing CLT shearwalls, with the aim of preventing the occurrence of brittle failure and ensuring adequate energy dissipation under seismic loads. Gavric et al. [4] indicated dissipative connections, such as fasteners (nails and screws) used in panel-to-panel connections, hold-downs and angle brackets, in which plastic hinges are developed, and non-dissipating elements, such as CLT panels and brackets and anchoring bolts attached to the base. The

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authors extended the proposal to multi-storey buildings, identifying panel-to-panel connections and hold-downs as dissipative connections, while requiring angle brackets to remain elastic. As a result of research efforts in Europe, a comprehensive CD approach was developed and proposed for inclusion in the next generation of Eurocode 8 [5]–[8]. Based on the proposal, CLT shearwall buildings can be classified into two levels of ductility, namely: medium ductility, which refers to buildings constructed from single panel or multi-panel CLT shearwalls that behave monolithically, and high ductility, which refers to multi-panel CLT shearwalls behaving in coupled-panel (CP) kinematic mode and where the panel-to-panel connections are designed to dissipate energy.

Casagrande et al. [9] proposed a CD approach for multi-panel CLT shearwalls using minimum potential energy method based on the developed analytical approach presented in [10]. Masroor et al. [11] extended the CD approach to include the bi-directional contribution of angle brackets, which was considered as a circular domain, as presented and investigated in [12]–[14]. A yield hierarchy was introduced for the dissipative connections, while ensuring that non-dissipative elements and those with limited-dissipative capability remain elastic. To achieve the proposed hierarchy and provide protection for non-dissipative elements, the concept of introducing over-strength factors were proposed and discussed.

The current study aims to validate the proposal developed in [11] by means of comparing the procedure to experimental test results and numerical models for multi-panel CLT shearwalls with conventional connections. The results of the experimental tests on connections are also presented, as they were used as inputs in the design equations and numerical models.

2 The CD design approach

2.1 General

This section reviews the CD approaches for multi-panel CLT shearwalls developed and reported in [11]. To ensure the proper sequence of the yield hierarchy, a CP kinematic mode is required, in which panel-to-panel connections yield before hold-downs. This behaviour implies that all panels have individual centers of rotation, which remain in contact with the ground. Figure 2 illustrates the investigated shearwall, consisting of m panels, subjected to gravity load, q, factored bending moment, M_f, and shear, V_f, at the base of the shearwall, and achieving CP kinematic behaviour. The figure also indicates the vertical (uplift) stiffnesses of the hold-down and shear stiffness in each panel-to-panel connection, $k_{h,z}$ and k_f, respectively, horizontal (shear) and vertical (uplift) stiffnesses of the angle bracket, $k_{a,x}$ and $k_{a,z}$, respectively, and the number of panel-to-panel connections and angle brackets used in each panel, n_f and n_a, respectively. The width and height of each panel are denoted as b and h, respectively.

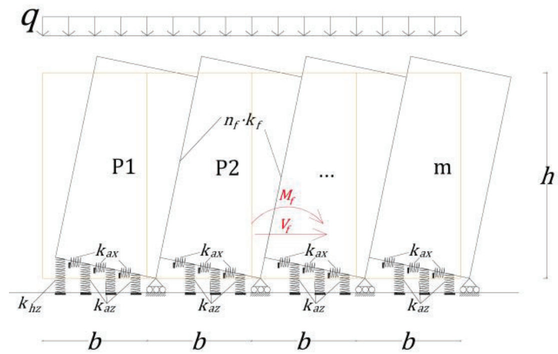

Figure 2: *The CP lateral behaviour of a multi-panel CLT shearwalls*

The proposal defines four categories of structural elements, as presented in Table 1: (1) Primary energy dissipative connections, which refers to panel-to-panel connections, considered to yield prior to other elements; (2) Other energy dissipative connections, which refers to hold-down connections, considered to yield after panel-to-panel connections; (3) Limited energy dissipative connections, which refers to angle brackets, considered elastic when panel-to-panel connections and hold-down have yielded; and (4) Non-energy dissipative elements, which refers to brittle structural elements such as CLT panels, which are expected to remain elastic until the energy dissipative connections reach a given load or displacement. Three over-strength factors were proposed to ensure such yield sequence is achieved and confirm that angle brackets and non-dissipative elements remain elastic. $\gamma_{r,h}$ is proposed to ensure that panel-to-panel connections yield prior to the hold-down, $\gamma_{r,a}$ ensures that angle brackets remain elastic when dissipative connections have yielded, and $\gamma_{r,ND}$ is meant to overprotect non-dissipative connections and ensure they remain elastic.

Table 1: *Proposed framework for the over-strength factors* [11]

Category	Element	Behaviour	Over-strength factor
Primary energy dissipative	Panel-to-panel connections	Yield	-
Other energy dissipative	Hold-down	Yield	$\gamma_{r,h}$
Limited energy dissipative	Angle brackets	Elastic	$\gamma_{r,a}$
Non-energy dissipative	E.g., CLT panels	Elastic	$\gamma_{r,ND}$

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2.2 Capacity-based design requirements

The first step to establish requirements for the CD approach involves achieving CP kinematic mode. This is ensured by satisfying the requirement presented in Equation (1), which is a function of the connections' stiffness contributing to rocking behaviour, wall geometry, gravity load, and bending moment acting at the base of the shearwall.

$$\frac{k_{h,z}}{n_f \cdot k_f} \geq \frac{1-\frac{\widetilde{M}\cdot[3\cdot m-2]}{m^2}}{\left(1+\frac{\beta\cdot n_a}{2}\right)-\frac{\widetilde{M}\cdot\left\{m\cdot\left(1+\frac{\beta\cdot n_a}{2}\right)-2\cdot\left[1+\frac{n_a\cdot(2\cdot n_a+1)\cdot\beta\cdot m}{6\cdot(n_a+1)}\right]\right\}}{m^2}} \qquad (1)$$

where, $\widetilde{M}$ and β are the dimensionless bending moment and vertical stiffness (uplift) ratio of mechanical anchors, presented in Equations (2) and (3), respectively.

$$\widetilde{M} = \frac{q\cdot m^2\cdot b^2}{2\cdot M_f} \qquad (2)$$

$$\varphi = \frac{k_{a,z}}{k_{h,z}} \qquad (3)$$

The requirement associated with the panel-to-panel connections yielding prior to the hold-down can be satisfied by ensuring the requirement outlined in Equation (4).

$$r_{h,z} \geq \gamma_{r,h} \cdot r_f \cdot \frac{k_{h,z}}{k_f} \qquad (4)$$

where $r_{h,z}$ and r_f are the yield resistance of the hold-down in the vertical direction (uplift) and the yield resistance of one panel-to-panel connection, respectively.

The angle brackets are required to remain elastic when energy-dissipative connections have yielded, which ensures that sliding is minimised. This can be done by satisfying the requirement in Equation (5).

$$\left[\frac{\gamma_{r,a}\cdot n_a\cdot\beta\cdot r_{h,z}}{r_{a,z}\cdot(n_a+1)}\right]^2 + \left(\frac{C_h\cdot V_f}{r_{a,x}\cdot m\cdot n_a}\right)^2 < 1 \qquad (5)$$

where, $r_{a,x}$ and $r_{a,z}$ are the yield resistance of one angle bracket in the horizontal (shear) and vertical (uplift) directions. V_f refers to applied shear acting at the base of the shearwall. C_h represents the ratio of the wall's bending moment resistance when hold-down yields to the bending moment acting at the base of the shearwall, and it can be obtained using Equation (6)

$$C_h = \frac{M^c_{r,h}}{M_f} \qquad (6)$$

The wall's bending moment resistance when hold-down yields can also be calculated using Equation (7)

$$M^c_{r,h} = b \cdot \left\{r_{h,z} \cdot \frac{[1+n_a\cdot(2\cdot n_a+1)\cdot\beta\cdot m]}{6\cdot(n_a+1)} + r_f \cdot n_f \cdot (m-1) + \frac{q\cdot m\cdot b}{2}\right\} \qquad (7)$$

The bending moment resistance of the shearwall, $M_{r,w}$, can be expressed as the wall's bending moment resistance when hold-down yields, $M^c_{r,h}$, which is required to be equal to or greater than the bending moment acting at the base of the shearwall due to the applied lateral loads.

$$M_{r,w} = M^c_{r,h} \geq M_f \qquad (8)$$

Finally, the non-dissipative elements are required to be capacity-protected. For example, the resistance of the CLT panels, $V_{r,CLT}$ is required to be equal to or greater than the applied shear load at the base of the shearwall when hold-down yields, considering the appropriate over-strength factor, as shown in Equation (9)

$$V_{f,CLT} = \gamma_{r,ND} \cdot C_h \cdot V_f \leq V_{r,CLT} \qquad (9)$$

3 EXPERIMENTAL TESTS RESULTS OF CONNECTIONS

The experimental tests were conducted on conventional connections used for CLT shearwalls. The average results of two tests on fully nailed WHT620 hold-downs [15] under uplift load and two tests on panel-to-panel connections consisting of partially threaded self-taping screws HBS 6x70 mm [16] are obtained from [17]. Figures 3 and 4 illustrate the load-displacement curves of the hold-down connections (HD-1 and HD-2) and panel-to-panel connections (PP-1 and PP-2), respectively. The average mechanical properties of each connection are presented in Table 2, based on the Equivalent Energy Elasto-Plastic (EEEP) curve, according to ASTM E2126-16 [18], including yield load, r_y, and corresponding displacement, d_y, elastic stiffness, k, and ultimate displacement, d_u.

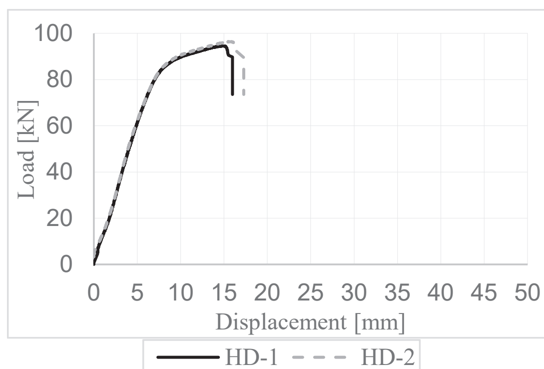

Figure 3: *Load-displacement curves of two tests on fully nailed WHT620 hold-down under uplift* [17]

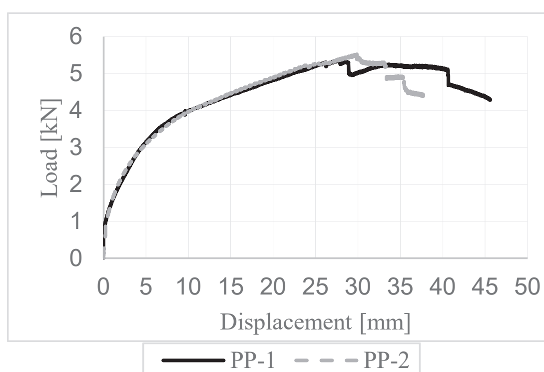

Figure 4: *Load-displacement curves of two tests on spline-joint panel-to-panel connections under shear* [17]

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Table 2: *The average of EEEP simulated mechanical properties of hold-down and panel-to-panel connections* [17]

Connection	r_y [kN]	d_y [mm]	k [kN/mm]	d_u [mm]
Hold-down	90.80	7.30	12.44	16.60
Panel-to-panel	4.60	5.50	0.84	41.60

The results of monotonic shear and uplift experimental tests on fully nailed TCN200 angle brackets without washers [19], are presented here in the following and shown in Figure 5. Two repeats were conducted for each loading. A displacement rate of 3 mm/min was selected, consistent with the rates used in tests on the hold-downs and panel-to-panel connections.

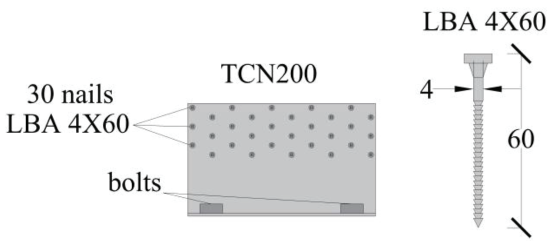

Figure 5: *Fully nailed TCN200 angle bracket*

Figures 6 and 7 illustrate the test setups for the angle bracket under monotonic uplift and shear loads, respectively. The CLT panels were composed of three layers, E1 grade, manufactured according to ANSI/APA (2020) [20]. Panel thickness was 105 mm (35-35-35), and the width of each board was 89 mm.

Figure 6: *Test set-up of angle bracket connection under uplift*

Figure 7: *Test set-up of angle bracket connection under uplift*

Figures 8 and 9 illustrate the load-displacement curves of the angle bracket connections under uplift (AU-1 and AU-2) and shear (AS-1 and AS-2), respectively. The average mechanical properties of the angle brackets are presented in Table 2, based on the EEEP curve.

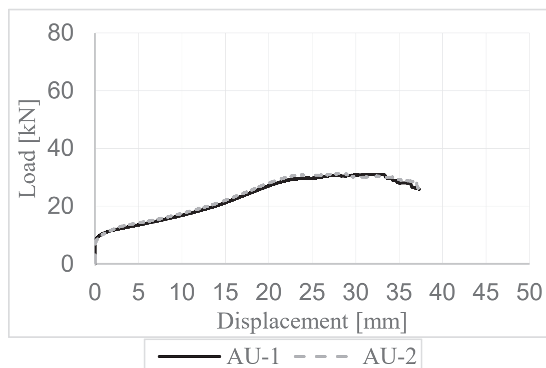

Figure 8: *Load-displacement curves of two tests on fully nailed TCN200 angle brackets under uplift*

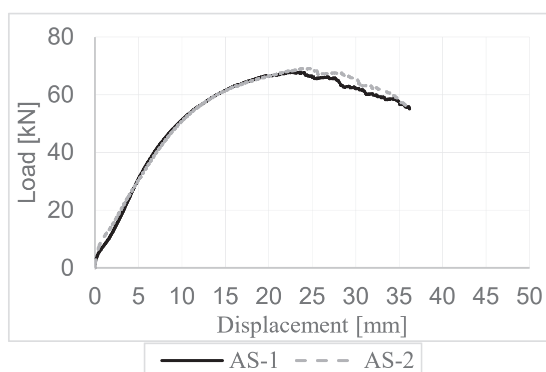

Figure 9: *Load-displacement curves of two tests on fully nailed TCN200 angle brackets under shear*

Table 3: *The average of EEEP simulated mechanical properties of TCN200 angle brackets under uplift and shear*

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Load	r_y [kN]	d_y [mm]	k [kN/mm]	d_u [mm]
Uplift	25.4	6.8	3.7	37.5
Shear	62.5	11.0	5.7	36.2

4 EVALUATION OF THE CD PROCEDURE FOR MULTI-PANEL CLT SHEARWALLS

Experimental tests were conducted on two CLT shearwalls (W-1 and W-2), with the configurations shown in Figure 10, to investigate the CD approach proposed in [11] and presented in section 2. Walls W-1 and W-2 were subjected to equivalent uniform gravity loads, q, of 1.45 kN/m and 6.67 kN/m, respectively. The shearwalls consisted of three panels with height and length of 2438 mm and 1219 mm, respectively. The walls were anchored to the steel base and to the CLT panel by means of fully nailed hold-down and angle brackets, (i.e., WHT620 and TCN200). Two hold-downs were used on each face of the wall, while two angle brackets were used in each panel attached only to one face. Panel-to-panel joints consisted of nine HBS 6x70 mm screws.

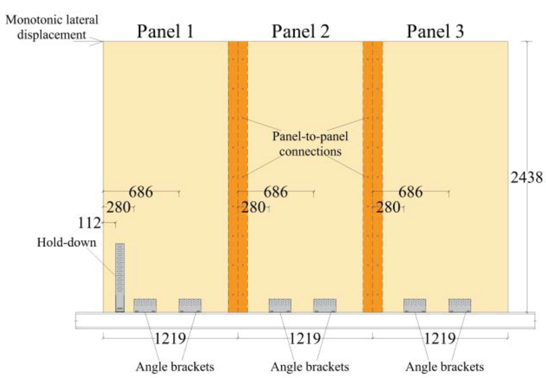

__Figure 10:__ Test set-up of CLT shearwalls

Numerical models were developed using SAP2000 software [21], as illustrated in Figure 11. Multilinear link elements were used to model the connections based on the curves obtained from the connection tests. The interaction between uplift and shear in angle brackets were modelled using macro element, as presented in [22]. A concentrated lateral load, F, was applied at the top of the shear-wall consistently with the position of the loading actuator in the experimental tests, while the gravity loads were applied on top of the panels. Rigid gap elements acting only in compression were modelled at the bottom of the walls to simulate the contact between the steel base and the CLT panels. At the top and bottom of each panel, diaphragm constraints were applied. Orthotropic material properties were used for the CLT panels. The input values of $E_{eff,1}$, $E_{eff,2}$, and G_{eff} were equal to 4100 MPa, 7900 MPa, and 361 MPa, respectively [17].

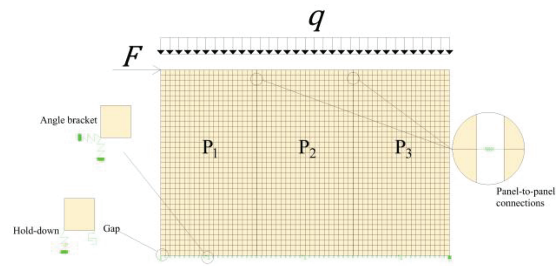

__Figure 11:__ SAP2000 numerical models of a three-panel CLT shearwalls

Figure 12 demonstrates the load-displacement curves obtained from experimental tests and numerical models. Also presented in the figure is the wall's shear resistance at the point of hold-down yielding, obtained by dividing the wall's bending moment resistance when hold-down yields (Equation (7)) by the wall height, h. For both walls, reasonable matches can be observed between the results obtained from experimental tests and numerical models. The calculated shear resistance obtained from Equation (7) appears to be lower than the peak resistance obtained from the experimental tests and numerical models. This additional strength could be attributed to the contribution of angle brackets after the yielding of the hold-down, which is represented in the tests and numerical models but not in the CD equations.

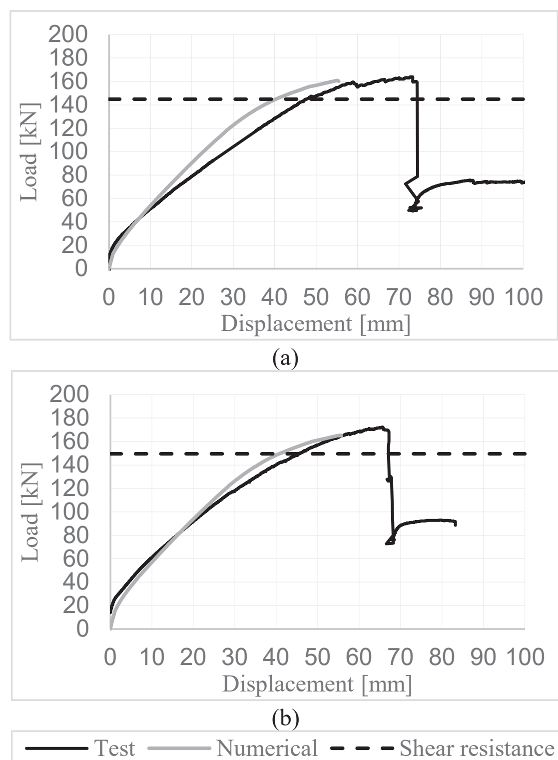

__Figure 12:__ Load-displacement curves of investigated CLT shearwalls: (a) wall W-1 and (b) wall W-2

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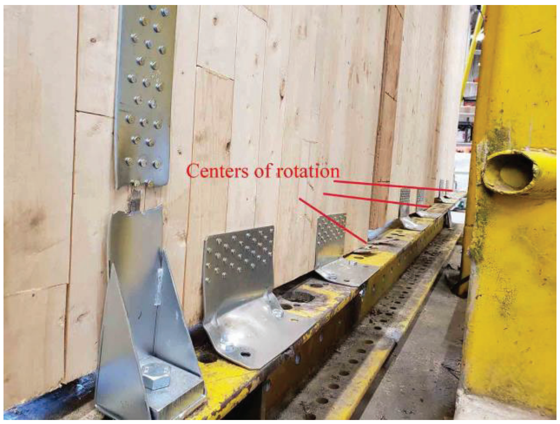
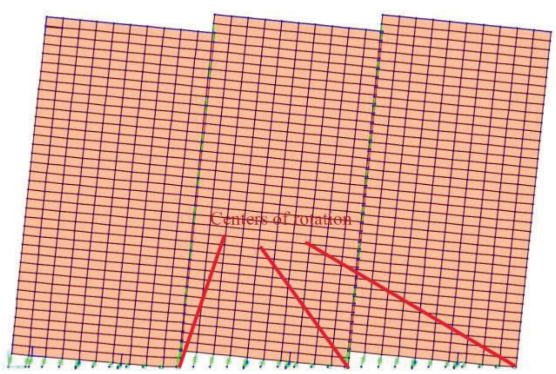
Evaluating the validity of the CD requirements, presented in Section 2.1, it can be noted that the CP kinematic mode was achieved in both experimental tests and numerical models, as shown in Figure 13(a) and (b), respectively. This satisfied the requirement outlined in Equation (1). The panel-to-panel connections yielded before hold-downs, which satisfied the requirement in Equation (4). The angle brackets remained elastic when hold-down yielded, satisfying the requirement in Equation (5), and the requirement related to the CLT panels remaining elastic was clearly also met.



(a)



(b)

Figure 12: *Load-displacement curves of investigated CLT shearwalls: (a) wall W-1 and (b) wall W-2*

Table 4 presents the results obtained from the CD expressions for each requirement, as outlined in Section 2.1. Due to the deterministic nature of these examples, overstrength factors of unity were employed for the hold-down, $\gamma_{r,h}$, and the angle brackets, $\gamma_{r,a}$. A value of 1.6 was set for non-dissipative elements, $\gamma_{r,ND}$, such as CLT panels, based on [4]. For each wall, the requirements outlined in Equations (1), (4), (5) and (9) were calculated with inputs obtained from table 2 and 3. As can be observed in the table, all requirements were satisfied for both walls. This demonstrates that satisfying the proposed equations could lead to the desired lateral behaviour in CLT shearwalls. It is also noteworthy to mention that calculating the shear resistance of CLT panels in order to compare it with the value obtained from Equation (9) is out of the scope of this research, however, observations from the experimental study clearly shows that the shear strength of the CLT panels far exceeded the shear forces induced by the lateral load.

Table 4: *CD requirements' values for walls W-1 and W-2*

Eq.	Values		Satisfied
	W-1	W-2	
(1)	3.28>0.85	$3.28 > 0.80$	Yes
(4)	181.60>135.81	181.60>135.81	Yes
(5)	0.65<1.00	0.66<1.00	Yes
(9)	231.62	239.24	-

5 CONCLUSIONS

The current study investigates the validity of a proposed capacity-based design procedure with the help of experimental tests and numerical models on multi-panel CLT shearwalls. The results of experimental tests of fully nailed TCN200 angle brackets without washers under uplift and shear loads were also presented. The mechanical properties of connections obtained from literature and those presented in the current study were used as inputs to the CD equations and numerical models. Experimental tests and numerical models from two CLT shearwalls comprised of the tested connections were evaluated. reasonable match was observed between the results obtained from experimental tests and numerical models, while wall shear resistance obtained from the capacity-based design equations was observed to be relatively lower. All the requirements from the capacity-based design approach were met in experimental tests, numerical modes and the results demonstrated that satisfying the proposed equations would lead to the desired lateral behaviour.

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