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In-Plane Strength and Stiffness of Cross-Laminated Timber Shear Walls

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Abstract: The research presented herein investigated the in-plane performance of cross-laminated timber (CLT) shear walls for platform-type buildings under lateral loading. Finite element models of CLT connections (i.e., brackets, hold-downs and self-tapping screws) were developed in OpenSees and calibrated against experimental tests to represent the connections' hysteresis behaviour under cyclic tension and shear loading. The results were incorporated into models of CLT single and coupled shear walls. The results in terms of peak displacement, peak load and energy dissipation were in good agreement when compared to full-scale shear wall tests. Subsequently, a parametric study of 56 single and 40 coupled CLT shear walls was conducted with varying numbers and types of connectors (wall-to-floor and wall-to-wall) for evaluating their seismic performance. It was found that the strength, stiffness and energy dissipation of the single and coupled CLT shear walls increased with an increase in the number of connectors. Single shear walls with hold-downs and brackets performed better under seismic loading compared to walls with brackets only. Similarly, coupled shear walls with four hold-downs performed better compared to walls with two hold-downs. Finally, ductility of coupled shear walls was found to be 31% higher compared to that of single shear walls. The findings from this research are useful for engineers to efficiently design CLT shear walls in platform-type construction.

Keywords: Finite Element Analyses; connection; ductility; parameter study; seismic performance

1. Introduction

1.1. Background

With a rise in sustainable practices, the construction industry is increasingly considering timber as a low-carbon footprint material for larger residential and non-residential applications. New mass timber products such as laminated veneer lumber (LVL) cross-laminated timber (CLT) [1], efficient connectors such as long self-tapping screws (STS) [2,3], innovative ductile connections [4,5], and pre-fabrication [6] have created an opportunity to build mid- to high-rise timber buildings.

CLT panels consist of several layers of boards stacked crosswise and glued together. A CLT element usually has three to nine glued layers of boards placed orthogonally to each other (90°) to form a solid panel [7]. Use of CLT for wall and floor panels offers many advantages: The cross-lamination itself provides improved dimensional stability and thermal insulation, and a fairly good response in case of fire, which are added benefits resulting from CLT's massiveness [8]. Furthermore, CLT is a clean product to work with, resulting in little waste or dust produced on site, which is preferable in terms of health and safety.

Platform-type construction, where each floor acts as a platform for the floor above, is the most common structural system used for low to mid-rise CLT buildings [8]. CLT shear walls, both single and/or coupled walls (Figure 1a–c), are commonly connected to the foundation or a concrete podium by steel brackets and hold-downs using metal fasteners like screws and nails (Figure 1d), while CLT walls and floors in the upper storeys are connected by brackets and hold-downs (HD) or by STS. The panels in between floors and walls are connected by screws using either lap or spline joints (Figure 1e–f). "Designing and building CLT structures, also in earthquake-prone regions is no longer a domain for early adopters, but is becoming a part of regular timber engineering practice" [9].

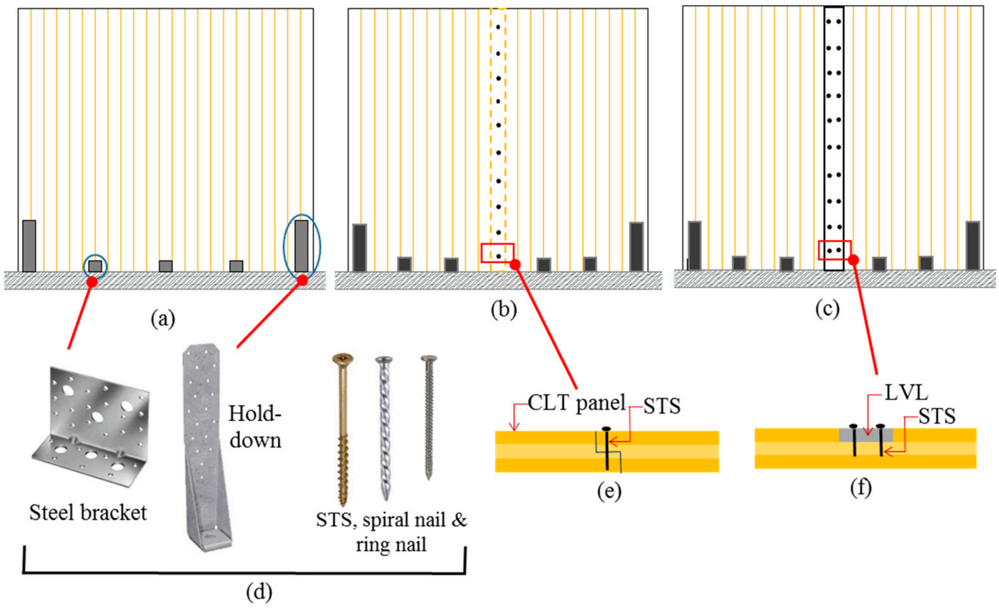

Figure 1. CLT shear walls: (**a**) single, (**b**) coupled wall with lap joint, (**c**) coupled wall with spline joint, (**d**) connectors and fasteners, (**e**) half-lap joint, (**f**) spline joint.

1.2. Impact of Connections on Performance of CLT Shear Walls

Since CLT panels are rigid in comparison to their connections, the stiffness of CLT systems mostly depends on the connections. Other aspects, such as openings cut into CLT panels, also affect the shear wall stiffness [10]. Tomasi and Smith [11] investigated the mechanical behaviour of angle brackets connecting CLT walls to the foundation and found that the connection's capacity depended on the geometries of the bracket (i.e., length and shape, presence of ribs and corrugation) and the fasteners connected to CLT panels (e.g., annular nails) and foundation (e.g., bolts).

Gavric et al. [12,13] conducted monotonic and cyclic tests on different hold-downs, brackets and shear connections for foundation, wall and -floor connections using 5-ply 85 mm-thick wall and 5-ply 142 mm-thick floor panels. The shear connections were tested in two configurations of half-lap and spline joints with STS of 8 × 80 mm and 8 × 140 mm. They observed that brackets have similar capacity and stiffness under tension and shear loading. In contrast, hold-downs showed higher strength and stiffness in tension when compared to bracket connections, while their shear resistance was negligible. The shear connections showed a ductile performance, and, for the chosen STS configurations, the half-lap joints exhibited higher strength and stiffness compared to the splines.

A similar study was conducted by Schneider et al. [14] on bracket connections using three types of fasteners: Spiral nails, ring nails and STS. They performed cyclic tests and developed a numerical model; the numerical results (ductility, stiffness and strength) showed good correlation with tests, however, the calculated damage index of the connections was found to be significantly different when compared to test results. Hossain et al. [3] evaluated the performance of CLT shear connections using

STS by conducting monotonic and cyclic tests on butt joints where the screws were installed at a doubly inclined angle between two panels acting in withdrawal. The results showed that highly ductile joints can be achieved.

Other studies focused on developing analytical models to estimate the resistance of single- and multi-panel CLT shear walls. Gavric and Popovski [15] proposed five models (D1–5): D1—assumed sliding only and that the resistance was equal to the shear resistance of the brackets; D2—assumed rocking and sliding with the resistance based on the shear resistance of the brackets and the overturning resistance of the HDs; D3—assumed pure wall rocking behaviour with the resistance being determined by accounting only for the uplift contribution of the connectors; D4—assumed sliding and rocking and that the brackets contributed to both shear and uplift resistance according to a quadratic interaction equation; and, D5—also assumed sliding and rocking with the brackets contributing to the shear and uplift resistances, but according to a linear interaction equation.

Flatscher and Schickhofer [16] applied a displacement-based approximation of connections' behaviour to describe the behaviour of CLT walls beyond the peak load at hand of a parameters study using experimentally verified finite element (FE)-simulations. They confirmed the distinct connection influence on the behaviour of CLT structures and suggested that initial slip values may essentially underestimate the actual wall deflections. It was further suggested that the application of vertical joints does not always improve the structural behaviour, as load-carrying capacity, stiffness and energy dissipation is decreased. Further, it was shown that the sole application of angle brackets may have a positive influence on load distribution compared to a combination of brackets with hold-downs [16].

Hysteretic models to capture the performance of vertical panel joints in CLT wall systems have been developed [17,18]. Tamagnone et al. [19] proposed a non-linear procedure for the seismic design of CLT wall systems whereupon a triangular compressive force at the wall-to-support interface was considered and the neutral axis calculated from an iterative procedure. Although wood is an orthotropic material, this model assumed wood to be a uniaxial elastic material. Furthermore, the model assumed that the displacement of the wall during rocking can happen in the negative Z-direction (e.g., whereby the wall corner can penetrate into the foundation) which is not a realistic scenario. Most recently, analytical models to determine the elastic stiffness and strength of multi-panel CLT shear walls based on the minimum potential energy approach were proposed by Casagrande et al. [20]. A kinematic model of multi-panel walls based on the stiffness ratio of HD and vertical shear connector between panels was used to study the elastic kinematic behaviour of a multi-panel CLT shear wall by adopting the dimensionless stiffness and load ratios to describe the shear wall's load-deformation path taken through different elastic behaviours under increasing lateral force. Furthermore, it should be noted that under the current draft stage for the new edition of Eurocode 5, coupled shear walls are considered to dissipate higher energy compared to single walls because they are connected via energy dissipative vertical shear connectors [21].

1.3. Performance of CLT Shear Walls in Platform-Type Construction

Understanding the actual behaviour of CLT shear walls under lateral loads is important for a reliable design of CLT buildings [22]. Popovski et al. [23] performed a series of quasi-static monotonic tests on CLT shear walls with 12 different configurations of wall-to-floor, wall-to-wall and storey-to-storey bracket connections, and demonstrated adequate seismic performance. Placing hold-downs on each end of the wall further improved the seismic performance. Gavric et al. [24] performed cyclic tests on both single and coupled CLT walls with three different wall configurations: (a) Single wall panels, (b) coupled wall panels with lap-joints, and (c) coupled wall panels with LVL spline joints. The walls were constructed with 85 mm-thick 5-ply CLT panels with angle brackets (BMF $90 \times 48 \times 3 \times 116$ mm) and hold-downs (HTT-22) used to connect the wall to the foundation using annular ring nails ($12 - 4 \times 60$ mm). 8×100 mm STSs were used for the wall-to-wall vertical joints. It was observed that the failure of the systems occurred at the connections, while the CLT wall panels were subjected to negligible in-plane deformations. Reynolds et al. [25] investigated the deformation and resistance of CLT shear walls for platform construction and confirmed that the movement of the shear walls was governed by rigid body movement of the panels. The authors further reported that the design methods currently used by structural engineers underestimate the resistance of the shear walls.

1.4. Objectives

The use of CLT in residential and non-residential buildings is becoming increasingly popular in North America. While the 2016 supplement to the Canadian Standard for Engineering Design in Wood, CSAO86 [26], provides provisions for CLT structures used in platform-type applications, it does not provide guidance for the in-plane stiffness and strength of CLT shear walls. Understanding the in-plane behaviour of CLT shear wall systems is essential for the reliable design of CLT buildings under lateral loads, yet no universal agreement has been reached for determining their in-plane stiffness and strength. The objective of this study was to quantify the strength, stiffness, ductility and energy dissipation capacity of CLT single and coupled shear walls with various types of connections for platform-type construction.

2. Finite Element Analysis

2.1. CLT Connections

Finite Element Analyses (FEA) models for different CLT connectors for wall configurations were developed based on previous research (Figure 1). Gavric et al. [12,13] performed tests on steel brackets (90 × 48 × 116 mm) with different fasteners, hold-downs using ring nails and double plane shear tests on CLT wall-to-wall, half-lap and spline joints using a different number of STS following the EN 12,512 [27] loading protocol. Schneider et al. [14] performed tests on steel brackets (90 × 48 × 116 mm) with four different fasteners (B_1 to B_4) applying a modified CUREE loading protocol [28]. The connections referenced are designated as B_1 to B_5 for brackets, HD_1 to HD_2 for hold-downs, and WW_1 to WW_2 for shear connectors throughout the article as shown in Table 1.

Table 1. CLT connections.

Connection Type	ID	Fasteners	Reference
Steel bracket 90 × 48 × 116 mm	B_1	18 16d SN 3.9 × 89 mm	[14]
	B_2	18 SFS screw 4 × 70 mm	[14]
	B_3	10 SFS screw 5 × 90 mm	[14]
	B_4	12 RN 3.8 × 76 mm	[14]
	B_5	11 RN 4 × 60 mm	[12]
Hold-down: HTT16	HD_1	9 RN 4 × 60 mm	[12]
Hold-down: HTT22	HD_2	12 RN 4 × 60 mm	[12]
Half-lap joint	WW_1	2 STS 8 × 80 mm	[13]
Spline joint	WW_2	4 STS 8 × 80 mm	[13]

CLT connections loaded in tension and shear (Figure 2a,d, respectively) were modelled in OpenSees [29] using zero-length spring elements as shown in Figure 2b,e and were defined by two nodes at the same location, where the nodes are connected by the uniaxial material model "Pinching4" [30] to represent the force-deformation relationship for the element. Pinching4 (Figure 2g) is a piecewise linear model with a "pinched" load-deformation response that accounts for strength and stiffness degradation under seismic loading [30]. The model accounted for the cyclic strength degradation between different cycles, as observed in the experimental tests. The model includes 16 parameters to define the backbone curve and unload-reload paths along with pinching behaviour. The least-square method was employed to estimate the parameters of the Pinching4 model from the backbone of the tested connections. The parameters were calculated based on the response of the forces at corresponding displacements from the backbone curve using equivalent energy elastic plastic (EEEP) curves according to ASTM E2126 [28]. An EEEP curve is a perfectly elastic plastic representation of the actual response of the specimen, which encompasses the same area of the experimental backbone curve from the origin to the ultimate displacement.

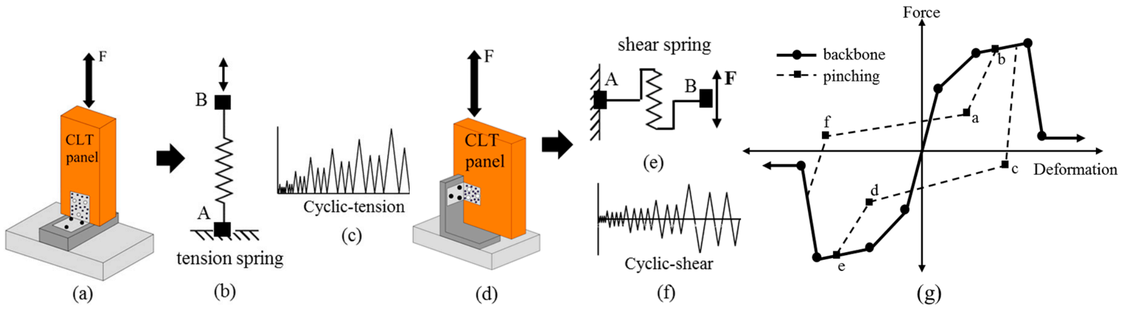

Figure 2. Cyclic tension test (**a**) schematic; (**b**) FE model and (**c**) loading protocol; cyclic shear test; (**d**) schematic; (**e**) FE model and (**f**) loading protocol; (**g**) Pinching4 model.

2.2. CLT Shear Walls

FEA models for CLT walls were developed based on research conducted at FPInnovations, Vancouver, Canada [23,24]. Popovski et al. [23] tested CLT panels made of 3-ply European spruce with a thickness of 94 mm (Figure 1a and Table 2). Steel brackets and hold-down connectors with various fasteners (annular ring nails, spiral nails, screws, and timber rivets) were used for the wall-to-foundation connections. The test setups for single and coupled CLT shear walls are shown in Figure 3a,b. Similarly, Gavric et al. [24] performed cyclic tests on three different wall configurations: (a) Single shear walls (panel of 3 m × 3 m); (b) coupled shear walls (two panels, each of 1.5 m × 3 m) with a half-lap joint; and, (c) coupled shear wall (two panels each of 1.5 m × 3 m) with an LVL spline joint (Table 2). Brackets and hold-downs with annular ring nails were used for wall-to-foundation connections and STSs were used for wall-to-wall vertical shear connections. As listed in Table 2, eight single CLT shear wall tests from Popovski et al. [23], and two single and six coupled wall tests from Gavric et al. [24] were used for the model validation.

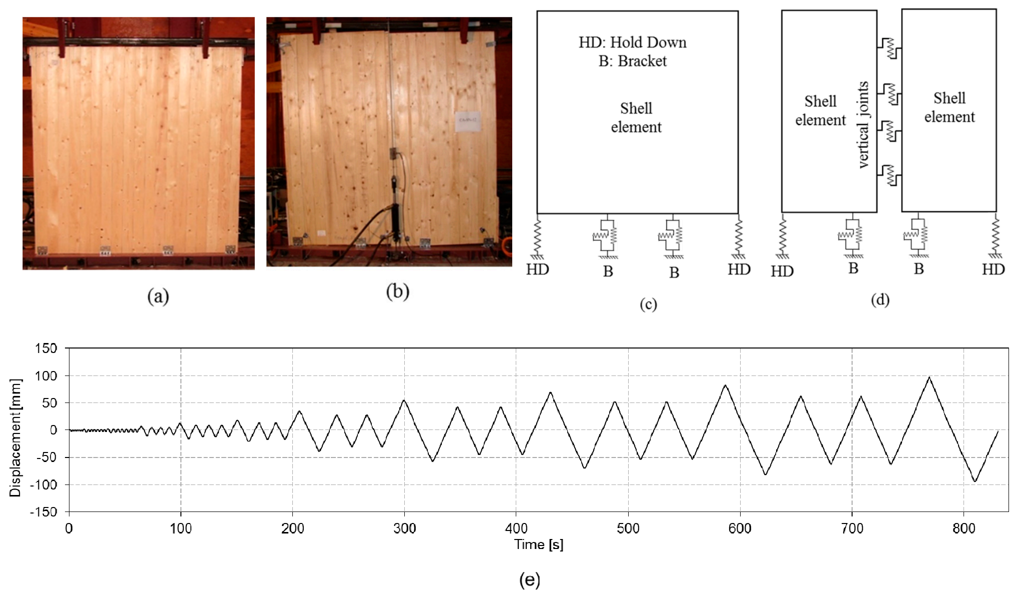

Figure 3. CLT shear wall's test setup ((**a**) single wall and (**b**) coupled wall); schematic of FE models ((**c**) single wall and (**d**) coupled wall); and, (**e**) CUREE loading [28].

Table 2. Experimental data set for single and coupled CLT shear walls.

Wall Type	Wall ID	Number (#) and Type of Brackets	# HDs (Type)	# Screws (Type)	Load (kN/m)	Reference
Single	SN-00	4 (B_1)	-	-	0.0	[23]
	SN-02	4 (B_1)	-	-	10.0	[23]
	SN-03	4 (B_1)	-	-	20.0	[23]
	S1-05	4 (B_2)	-	-	20.0	[23]
	S2-06	4 (B_3)	-	-	20.0	[23]
	RN-04	4 (B_4)	-	-	20.0	[23]
	SN-20	7 (B_1)	-	-	20.0	[23]
	SNH-08	3 (B_1)	2 (HD_1)	-	20.0	[23]
	I.1	2 (B_5)	2 (HD_2)	-	18.5	[24]
	I.2	4 (B_5)	2 (HD_2)	-	18.5	[24]
Coupled	II.1	4 (B_5)	2 (HD_2)	20 (WW_1)	18.5	[24]
	II.3	4 (B_5)	2 (HD_2)	10 (WW_1)	18.5	[24]
	II.4	4 (B_5)	4 (HD_2)	5 (WW_1)	18.5	[24]
	III.1	4 (B_5)	2 (HD_2)	2 × 20 (WW_2)	18.5	[24]
	III.2	4 (B_5)	2 (HD_2)	2 × 10 (WW_2)	18.5	[24]
	III.3	4 (B_5)	4 (HD_2)	2 × 5 (WW_2)	18.5	[24]

FEA models of CLT single and coupled shear walls were developed in OpenSees (Figure 3c,d). These tests demonstrated that CLT panels acted as a rigid body and all non-linear deformation occurred at the connections [18]. To represent the actual kinematic behaviour of the shear walls, the CLT panels were modelled using plane-stress shell elements with elastic material properties (as listed in Table 3) and the metal connectors were modelled using non-linear zero-length springs with the Pinching4 hysteresis model as shown in Figure 2g.

Both Gavric's and Schneider's tests [12,14] showed that bracket connections have similar resistance to cyclic shear and tension (Table 2). Therefore, each bracket was modelled with two-orthogonal zero-length springs at the same location (Figure 3c,d) and calibrated from connection tests as described in the previous section. The orthogonal zero-length springs simulate the sliding and rocking of the shear walls. In contrast, the hold-downs were modelled using only a single zero-length spring to resist rocking by neglecting their shear resistance as observed by Gavric et al. [12]. In the coupled shear wall FEA (Figure 3d), the vertical shear connectors for both half-lap and spline joints were modelled using two-node non-linear spring Pinching4 elements. The calibrated models for connections (brackets, hold-downs and STSs screw connections) were then incorporated in the CLT shear wall model.

Table 3. CLT material properties.

Elastic Modulus (MPa)			Shear Modulus (MPa)			Poisson's Ratio		
E_x	E_y	E_z	G_{xy}	G_{yz}	G_{zx}	υ_{xy}	υ_{yz}	υ_{zx}
11,700	9000	1000	563	731	100	0.35	0.07	0.35

2.3. Parametric Study for Single and Coupled CLT Walls

A parametric study was performed on single and coupled CLT shear walls with variations in the number and types of brackets and hold-downs. Two types of single shear walls were considered: CLT shear walls with brackets only (Case A), and CLT shear walls with both brackets and hold-downs (Case B). The CLT panels were 2.3 m × 2.3 m, 3-ply and 94 mm thick. For Case A, the shear walls with brackets were analyzed with five different types of fasteners (B_1 to B_5; Table 1), with the number of brackets varying from 4 to 7. Where walls were connected by brackets and hold-downs (Case B), two types of hold-downs (HD_1 or HD_2) were considered at the end of wall-to-floor interfaces, with the number of brackets varying from 2 to 5. Therefore, the total number of connectors remained the same for both Case A and Case B type walls.

In the parametric study on coupled CLT shear walls, the number and types of brackets (B_1 to B_5), hold-downs (HD_1 to HD_2) and vertical joints (WW_1 to WW_2) was varied. Two 1.15 m × 2.3 m CLT panels (3-ply, 94 mm thick, and total wall size same as single wall size of 2.3 m × 2.3 m) were connected by vertical joints as shown in Figure 1b,c. Two types of coupled shear walls were considered: CLT coupled shear wall with 2 hold-downs (2HDs) at the outer edges of each panel (Case C); and CLT coupled shear walls with 4 hold-downs (4HDs), both at the outer and inner edges of each panel (Case D). Each wall had a total of 4 brackets (2 on each side of the panel) connected with either half-lap joints (WW_1; 20 screws in one row) or spline joints (WW_2; 20 screws in two rows of ten). Reversed cyclic displacements were applied in the top nodes of the FEA wall models following CUREE loading protocol (Figure 3e). Table 4 summarizes the parameters.

Table 4. Overview of parameter study.

Wall Type	Single Walls		Coupled Walls	
Connector	Case A (brackets B_1 to B_5)	Case B (HD_1 or HD_2)	Case C (2HDs) (brackets B_1 to B_5)	Case D (4HDs) (brackets B_1 to B_5)
# Brackets	4, 5, 6, 7	2, 3, 4, 5	2 (half-lap joint)	2 (spline joint)

3. Results and Discussion

3.1. CLT Connections

The FEA models of connections were calibrated using the test results presented by Gavric et al. [12,13] and Schneider et al. [14]. Figure 4a,b show the hysteresis of bracket B_1 under shear and tension, respectively, Figure 4c shows the hysteresis of hold-down HD_1 under tension, and Figure 4d shows the hysteresis of STS half-lap joint WW_1 under shear. The Pinching4 model accurately captured the load-deformation hysteresis behaviour, including the backbone curves of CLT connections, with the hysteresis loops showing that the FEA line up with the test results in most of the cycles. However, in some cycles, the test results show irregular drops or changes in slope as a result of sudden fastener failures or formation of cracks and wood failure that was not captured by the FEA models.

From the FEA, the connections' stiffness, ductility, yield and ultimate load, and yield and ultimate displacement were calculated. All parameters as listed in Table 5 (ultimate load (P_u), ultimate displacement (d_u), yield load (P_y), yield displacement (d_y), ductility (D) and elastic stiffness (K_e)) were computed based on EEEP curves. The peak loads (P_{peak}) and the energy dissipation capacity (E) from the hysteresis loops are also shown in Table 5. From these results, it can be shown that the brackets have similar strength, stiffness and ductility under both tension and shear tests; therefore, they can be utilized for designing for both shear and tension resistance. The differences between the FEA models and experiments of Gavric et al. [12,13] and Schneider et al. [14] were found to be, on average, 7% and 12% for peak loads and energy dissipation, respectively. These differences can be considered acceptable and therefore, the connection models may be deemed appropriate to be implemented into the full-scale CLT shear wall model to validate these experimental results.

Table 5. CLT Connection FE results and parameters from EEEP curve.

Type of Test	ID	P_u (kN)	d_u (mm)	P_y (kN)	d_y (mm)	D (–)	K_e (kN/mm)	P_{peak} [1] (kN)	P_{peak} [2] (kN)	E [1] (kNm)	E [2] (kNm)
Tension	B_1	39.5	29.6	45.4	6.9	4.3	6.6	51.9	51.0	6.2	6.0
	B_2	43.0	27.9	48.0	8.1	3.4	5.9	52.1	53.9	4.4	4.1
	B_3	35.0	23.4	37.3	7.8	3.0	4.8	44.3	44.3	2.5	2.7
	B_4	31.6	22.8	35.2	4.6	5.0	7.7	39.9	42.2	2.8	2.5
	B_5	20.2	23.1	20.2	8.1	2.9	2.5	25.8	24.2	0.9	1.0
	HD_1	30.0	23.2	32.4	12.9	1.8	2.5	38.0	36.8	1.1	1.0
	HD_2	40.8	22.0	41.8	8.5	2.6	4.9	51.9	50.2	1.3	1.4
Shear	B_1	33.7	33.2	38.1	7.7	4.3	5.0	41.5	44.8	6.5	7.2
	B_2	42.1	32.4	50.0	9.4	3.5	5.9	53.4	54.8	8.5	8.3
	B_3	41.6	33.7	43.0	10.7	3.2	4.2	51.9	50.6	8.6	8.2
	B_4	34.6	29.5	37.3	6.9	4.4	5.6	43.3	42.1	6.6	5.9
	B_5	20.9	35.4	23.9	11.2	3.2	2.2	26.0	27.8	4.8	5.2
	WW_1 [3]	2.2	31.1	2.2	6.4	4.9	0.4	2.7	2.7	0.5	0.5
	WW_2 [3]	1.6	47.9	1.7	9.7	5.0	0.2	2.0	2.0	0.6	0.5

Note: [1] FE results, [2] test results; [3] calculations are based on single connector/shear plane.

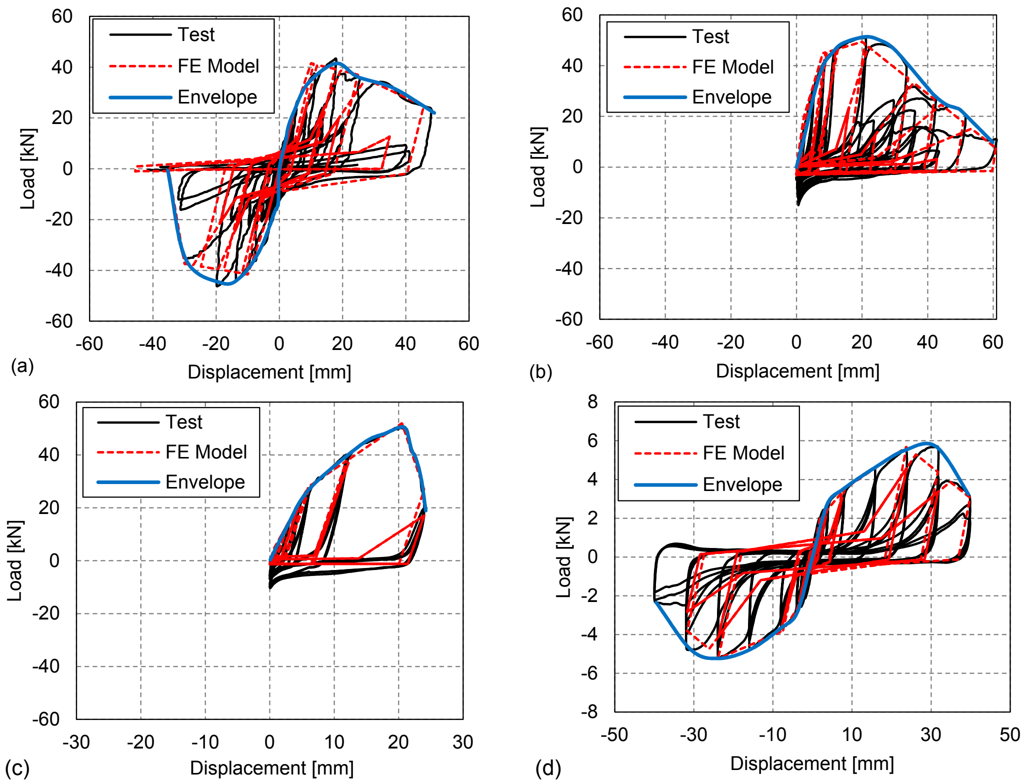

Figure 4. CLT connection's test vs. FEA model: (**a**) Bracket B_1- shear; (**b**) bracket B_1- tension; (**c**) hold-down HD_1-tension, and (**d**) STS connector WW_1-shear (two connectors).

3.2. CLT Shear Walls

The calibrated connection models were incorporated into the FEA models of the CLT shear walls and the wall models were analyzed using the loading protocols followed in the full-scale tests of Popovski et al. [18] and Gavric et al. [19]. Similar to these tests, reverse cyclic displacements were applied in the top nodes of the FEA wall models, and the resulting force was measured in each cycle at the bottom of the wall support. The load-displacement graphs from the tests and FEA models were plotted together, and the accuracy of the FEA models was compared and validated against the CLT shear wall test results as presented in Figure 5. The hysteresis loops along with their backbone curves indicate a strong agreement between tests and FEA.

The parameters representing the shear wall seismic performance (stiffness, strength and ductility) were calculated from the backbone curves based on EEEP curves according to ASTM E2126 [25] (Table 6). Overall, the peak and ultimate loads and displacements, energy dissipations and stiffness of the walls increased with the increasing number of connectors. For example, the single wall with four brackets and two HDs (I.1) showed 42% and 48% higher peak loads and displacements, respectively, compared to the single wall with two brackets and two HDs (I.2). Moreover, with the higher gravity loading, peak and ultimate loads, energy dissipation and stiffness all increased. However, the change in ductility was not found to be significant and no clear trend was observed.

In the coupled shear walls, the walls with four HDs showed better seismic performance compared to walls with two HDs in terms of peak and ultimate loads, and displacements and energy dissipation. Additionally, with the same number of screws in vertical connections, the walls with half-lap joints showed higher performance in terms of peak and ultimate loads compared to walls with spline joints. Most importantly, the change in ductility of the walls with half-lap joints was found to be significant (i.e., 36% higher compared to walls with spline joints).

As seen in Table 6, the average differences between FEA and tests on single CLT walls' tests [18] in the peak loads, peak displacements and energy dissipation capacities were 5%, 10% and 6%, respectively. Similarly, by comparing FEA to tests on coupled CLT walls, the average differences in the peak loads, peak displacements and energy dissipation capacities were found to be 4%, 7% and 11%, respectively. The differences were small and can be considered as acceptable.

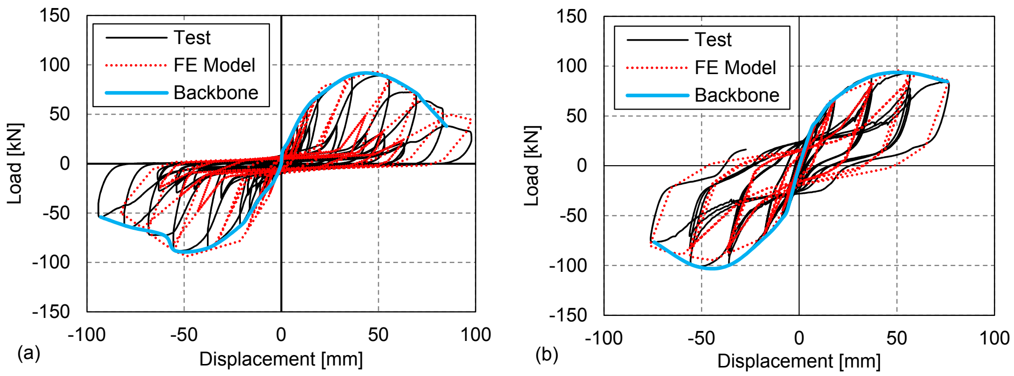

Figure 5. CLT shear wall FEA vs. test: (**a**) Single shear wall (SN-00) and (**b**) coupled shear wall (II.1).

Table 6. CLT single and coupled shear walls parameters from EEEP curve and FEA.

ID	P_{peak} [1]	P_{peak} [2]	d_{peak} [1]	d_{peak} [2]	E [1]	E [2]	P_u [1]	d_y [1]	D [1]	K_e [1]
	(kN)	**(kN)**	**(mm)**	**(mm)**	**(kNm)**	**(kNm)**	**(kN)**	**(mm)**	**(–)**	**(kN/mm)**
SN-00	93.3	88.9	48.3	44.9	26.4	27.8	73.2	19.0	3.5	4.4
SN-02	96.4	90.3	40.2	41.7	28.8	30.5	74.9	17.8	3.9	4.7
SN-03	99.6	98.1	46.2	44.1	29.9	31.0	78.9	17.8	4.5	4.9
S1-05	97.8	102.7	31.8	35.3	25.6	28.1	79.7	18.6	3.3	4.9
S2-06	92.9	100.1	34.8	42.2	25.0	26.9	76.4	19.2	3.3	4.6
RN-04	99.3	102.3	35.7	39.2	25.6	26.8	79.6	16.6	3.4	5.4
SN-20	153.9	152.1	47.4	40.9	44.3	45.5	124.9	19.4	4.2	7.2
SNH-08	126.2	118.2	49.8	53.1	36.4	37.7	99.5	14.3	3.9	7.3
I.1	75.0	70.7	34.7	38.7	12.6	13.1	59.7	12.6	3.0	5.0
I.2	106.7	104.2	51.5	57.3	21.9	24.1	86.2	16.2	3.5	5.9
II.1	95.1	97.2	47.9	53.4	30.0	32.0	74.9	15.7	5.1	5.3
II.3	84.1	84.4	50.5	46.6	26.9	28.4	66.2	14.9	5.6	5.0
II.4	100.8	93.1	57.8	53.8	21.1	24.5	80.1	21.3	3.7	4.2
III.1	94.6	102.5	70.0	66.0	30.8	33.1	75.4	18.5	4.5	4.6
III.2	88.2	91.8	68.6	65.4	27.3	25.5	68.6	19.5	4.3	3.9
III.3	101.7	102.9	56.1	54.3	21.1	28.0	81.0	22.3	3.6	4.0

Note: [1] FE results, [2] test results.

3.3. Parametric Study

The results from the parametric study on single walls, illustrated in Figure 6, demonstrate that the capacity, stiffness and energy dissipation increase with an increase in the number of connectors. By increasing the connectors from four to seven, the average strength, stiffness and energy dissipation in single shear walls with brackets increased by 57%, 39%, and 30%, respectively (Figure 6a,b,d). Similarly, the increase in average strength, stiffness and energy dissipation in the shear walls with brackets and hold-downs was 53%, 33%, and 39%, respectively (Figure 6e,f,h). While the increase in strength in single shear walls was found to be linear with the increase in the number of connectors, the increase in stiffness and energy dissipation did not follow a linear trend. However, placing the HDs on the corner of the single walls increased the seismic performance significantly (i.e., the increase in stiffness and energy dissipation was found to be 49% and 23%, respectively). There was no clear trend observed in the change in ductility in the shear walls, however, the walls with HDs showed higher ductility compared to walls with brackets only (i.e., 7% higher ductility).

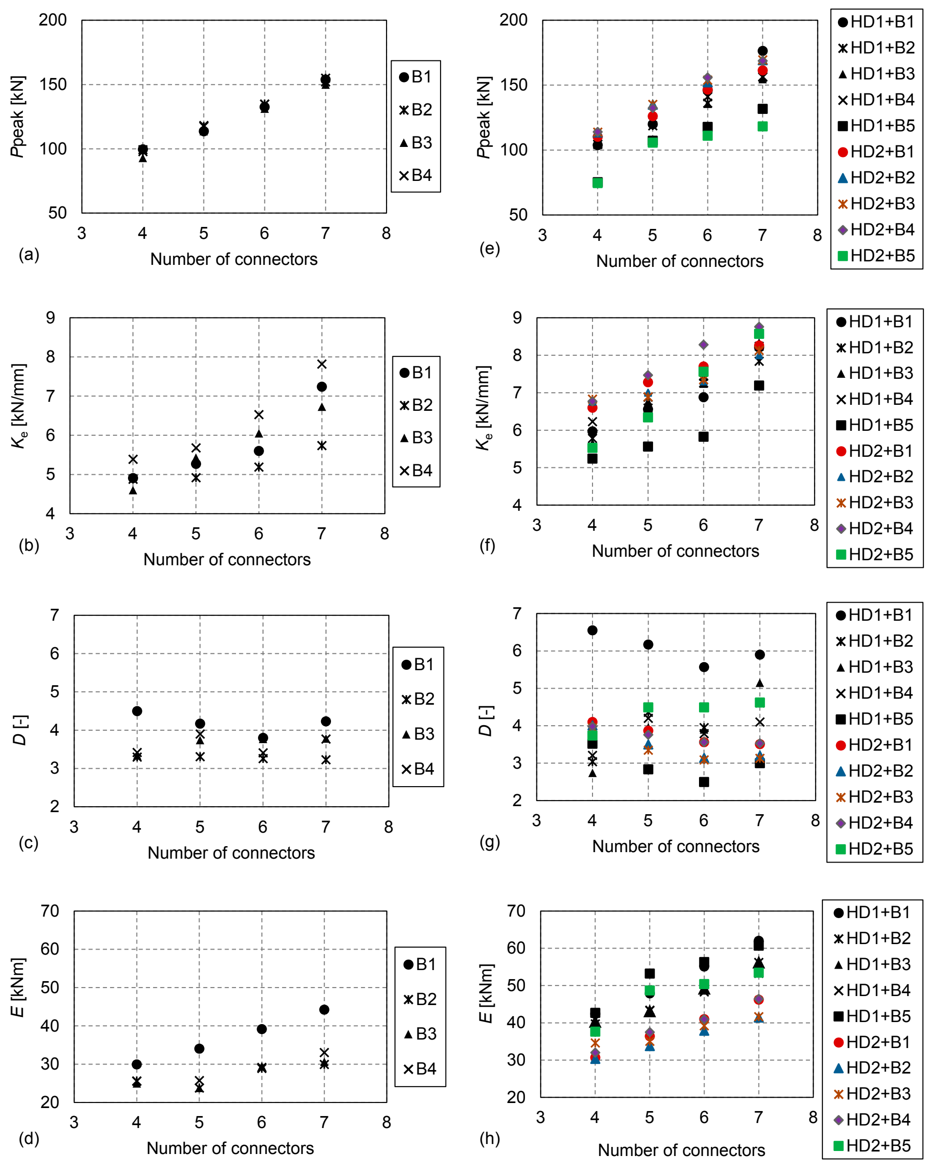

Figure 6. Single walls with brackets: (**a**) capacity, (**b**) stiffness, (**c**) ductility, and (**d**) energy. Single walls with brackets and HDs: (**e**) capacity, (**f**) stiffness, (**g**) ductility, and (**h**) energy.

The results from the parametric study on coupled CLT shear walls are shown in Figure 7. By comparing the same number of connectors in the vertical shear connections of the coupled shear walls, it was observed that the walls with half-lap joints performed better compared to walls with spline joints. The average strength, stiffness, ductility and energy dissipation in the coupled shear walls with half-lap joints was found to be 16%, 32%, 10%, and 18% higher, respectively, when compared to walls with spline joints (Figure 7a,b,d). Furthermore, the coupled shear walls with four HDs showed higher strength, stiffness and energy dissipation (i.e., 43%, 25%, and 14% higher, respectively) when compared to the coupled shear walls with two HDs (Figure 7e,f,h). The coupled shear walls with the HD_2 type of hold-down showed higher seismic performance in terms of strength, stiffness and energy (i.e., 12%, 8% and 26% higher, respectively) compared to shear walls with the HD_1 type of hold-down. By contrast, the ductility decreased with an increase in the number of HDs by 20%. However, the average ductility of the coupled shear walls was found to be 5.1, some 31% higher than the single CLT shear wall's average of 3.9.

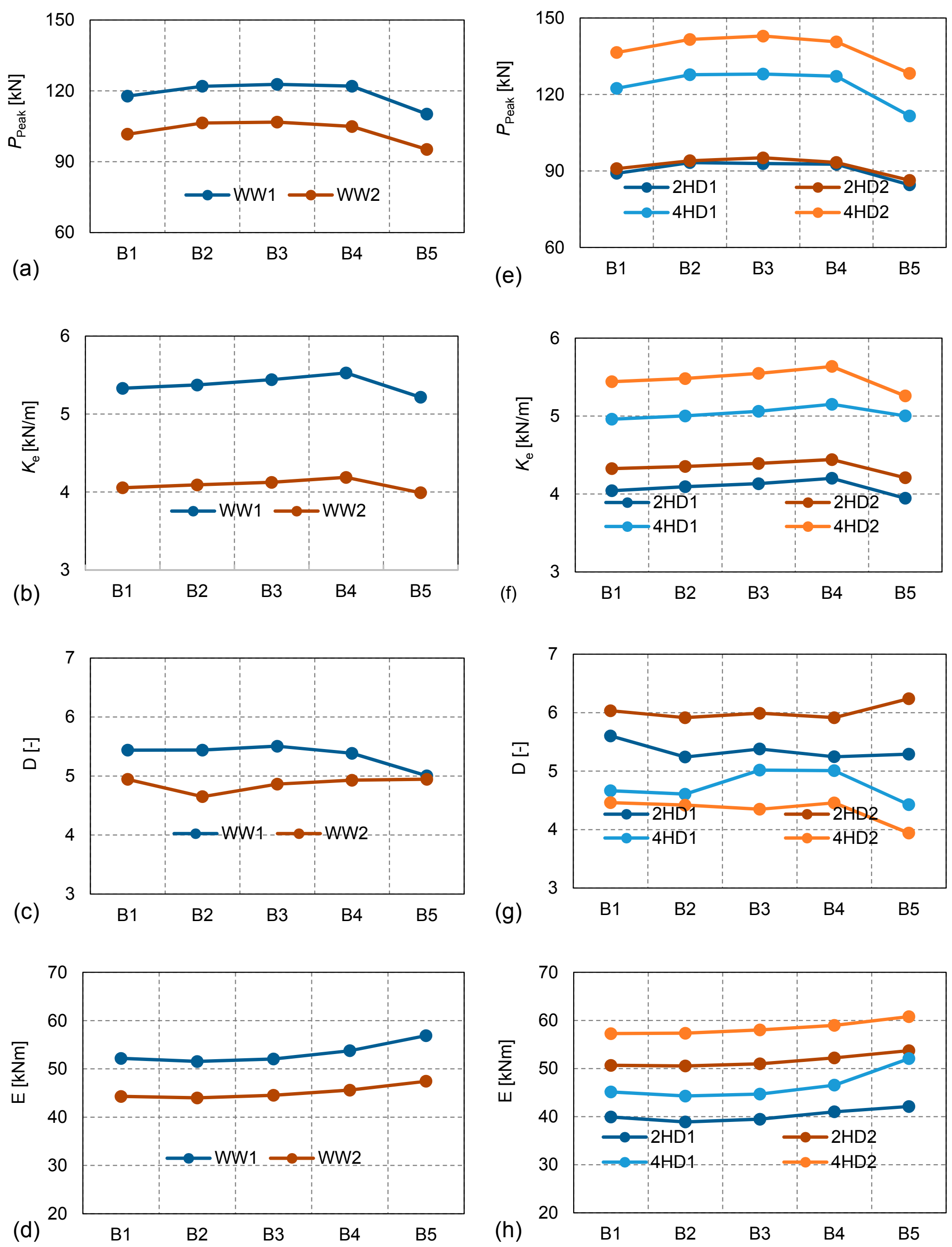

Figure 7. Coupled CLT shear walls with half-lap vs. spline joints: (**a**) capacity, (**b**) stiffness, (**c**) ductility, and (**d**) energy. Coupled CLT shear walls with two HDs vs. four HDs: (**e**) capacity, (**f**) stiffness, (**g**) ductility, and (**h**) energy.

4. Conclusions

The present study evaluated the behaviour of single and coupled CLT shear walls under lateral loading. FEA models of CLT connections were developed using nonlinear springs which were calibrated against test results and utilized to model full-scale shear walls under reversed cyclic loading. The FEA models of the CLT shear walls were compared to and verified against published test results. Subsequently, a parametric study was conducted with variations in the number and type of connectors evaluating the capacity, stiffness, ductility and energy dissipation for both single and coupled CLT shear walls. The investigations allowed the following conclusions to be drawn:

(1) The FEA models using the Pinching4 element accurately predicted the hysteresis behaviour of CLT connectors (e.g., brackets, hold-downs and shear screws).
(2) The FEA models of CLT shear walls closely predicted the load-deformation curves and the energy dissipation capacities of the shear walls when compared to published test results.
(3) It was observed that the capacity, stiffness and energy dissipation of the single and coupled CLT shear walls increases with the increase in the number of connectors.
(4) Ductility in the coupled shear walls was found to be 31% higher than in single shear walls. The decrease in ductility with an increase in the number of connectors was not significant.
(5) Single shear walls with hold-downs and brackets performed better under seismic loading compared to walls with brackets only (23% higher stiffness, 49% more energy dissipation).
(6) Coupled shear walls with four HDs performed better compared to coupled shear walls with two HDs (e.g., 43%, 25%, and 14% higher capacity, stiffness and energy dissipation, respectively, observed).
(7) Coupled shear walls with half-lap joints performed better under seismic loading compared to walls with spline joints.

In order to efficiently design CLT buildings, it is important to accurately predict the strength and stiffness of both single and coupled CLT shear walls with various types and numbers of connectors. The findings from this research will be a useful tool for engineers to efficiently design CLT shear walls in platform-type construction.

The findings from this study were based on the CLT connections that were available and used in traditional timber construction; however, a future study aims to investigate CLT shear walls with high performance connections that dissipate high seismic energy. The present study focused on CLT coupled walls with two panels only. Future research will focus on CLT coupled walls with more than two panels connected vertically. Furthermore, this research considered a constant vertical load of 20 kN/m on the CLT shear wall, which corresponds to a realistic loading for mid-rise CLT platform-type buildings. As this vertical load has an impact on the resistance and deflection of CLT shear walls, future work should focus on this parameter, in addition to top joints and perpendicular walls.

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