Lateral Behaviour of Cross Laminated Timber Shear Walls under Reversed Cyclic Loads

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ABSTRACT: Cross laminated timber (CLT) shear walls typically consist of solid engineered timber panels connected by metal hardware such as hold-downs, angle-brackets and others. Under seismic loads, the panel elements deform mainly in a rocking mode coupled with a sliding mode and small amount of in-plane bending/shear deformations. The connection system normally governs the lateral behaviour of CLT shear walls. This paper presents a finite element wall model CLTWALL2D to study CLT shear wall behaviour. The model consists of elastic orthotropic plate elements for the panels and nonlinear spring elements for the connections. Contact elements are also used for the panel-to-panel interactions. The nonlinear spring properties are represented by a subroutine called HYST that is able to model the strength and stiffness degradation and the pinching effect commonly observed in timber connections. The HYST parameters are calibrated by experimental data of CLT connections and embedded to the CLTWALL2D model. The wall model is validated against experimental data of a CLT shear wall test database. Parametric studies are then carried out to study the influence of gravity loads and vertical connection densities on the wall behaviour in terms of strength, stiffness, ductility, and energy dissipation.

1. INTRODUCTION

As an innovative engineered wood product, cross laminated timber (CLT) provides a competitive alterative solution for multi-storey buildings. In Europe and Australia, up to 10-story CLT buildings have been built in the last decade. Although not included in NZS3603 timber design standard, CLT buildings are also being constructed in New Zealand by referencing knowledge from overseas. When CLT buildings are built in a seismic prone country like New Zealand, it is important to have a thorough understanding of their seismic performance and sufficient technical information will be required for future update of the timber standard. Seismic performance of CLT systems has gained a lot of interests among researchers around the world. A number of experimental studies have been carried out on CLT systems. For example, in the SOFIE project led by IVALSA in Italy (Ceccotti, et al. 2006 & 2013), a comprehensive test database was established on CLT connections, shear walls, and multi-story buildings including a cap-stone shake table test on a full-scale 7-story CLT building. Researchers in Canada and Japan have also conducted a series of wall tests and full-scale building tests with the focus on the CLT products manufactured by domestic wood species.

Similar to light timber-framed buildings, CLT buildings rely on shear walls to provide seismic resistance. Unlike light timber-framed shear walls, CLT panels have very high in-plane strength and stiffness so that the wall lateral behaviour is governed by the connection systems. Under lateral loads, the wall elements tends to have a rocking mode and a sliding mode while the in-plane deformations of the wall elements are very small. It is therefore critical to understand the influence of the connections on the wall behaviour. A limited number of research has been carried out on the connection behaviour. For example, Schneider et al. (2012) tested over 100 CLT connections. The current connection technology for CLT buildings mainly follows the European practice. Off the shelf metal fasteners/connectors such as hold-downs, self-tapping screws, lag screws, nails, and steel brackets are commonly used.

Numerical models have been developed in commercial finite element software packages to study the behaviour of CLT shear walls or buildings. For example, a component-based finite element CLT wall model has been developed in ABAQUS by Rinaldin et al. (2013). Simplified analytical models have also been developed with the assumption of rocking-only rigid wall elements (Pei, et al. 2013). For the

numerical models and the analytical models, robust connection elements are required in order to reasonably capture the hysteretic characteristics of the CLT shear walls such as strength/stiffness degradations and the pinching effect. The multi-linear connection springs used in most commercial software packages have limitations because the load-deformation hysteresis of a typical timber connection shows nonlinearity in the entire loading history and sometimes it is challenging to define a yielding point. Therefore, in this study, a mechanics-based subroutine called HYST is used to model the hysteretic behaviour of the connections. The model parameters are calibrated by fitting the backbone curve of the connection hysteresis. The calibrated HYST are then incorporated into a wall model called CLTWALL2D to simulate the CLT wall behaviour. CLT shear wall test results will be used to check the wall model prediction accuracy. With validated CLTWALL2D, parametric studies will be further carried out to study the influence of gravity loads, and connection densities on the CLT shear wall performance.

2. CLTWALL2D MODEL DESCRIPTION

CLTWALL2D model has been developed in FORTRAN programming language and compiled by Intel Visual Fortran Compiler v10.0. As shown in Fig. 1, it consists of three types of elements: 1) 2D plate elements for CLT panels; 2) HYST elements for hold-down (HD) connections, angle bracket (AB) connections and lap joints with self-tapping screws between panels if applicable; and 3) contact elements between CLT panels. The formulations of the 2D plate elements and the HYST subroutines have been introduced in literature (Foschi, 2000 and Li, et al. 2012). The plate elements only consider in-plane behaviour of the panels with linear elastic wood properties assumed. The HYST elements are nonlinear springs with the $F(\Delta)$ response calculated by embedded finite element subroutines shown in Fig. 1 as well. The HYST subroutine calculates the connection behaviour along horizontal and vertical directions. Rotational strength and stiffness of these connections are not considered since their influence on wall behaviour is negligible.
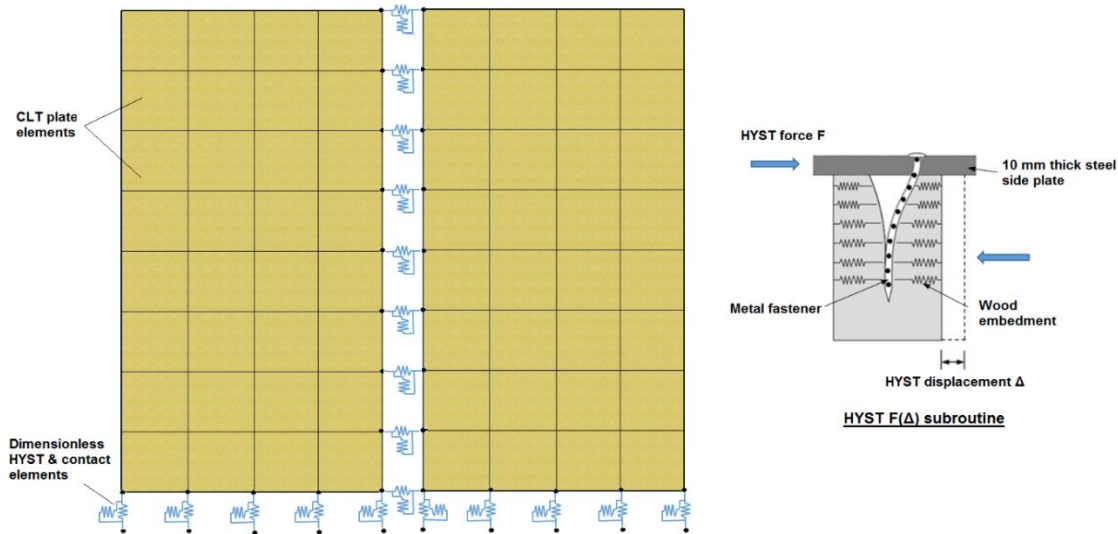

Figure 1 Schematics of CLTWALL2D and HYST subroutine

HYST is a mechanics-based model originally developed to simulate the hysteretic behaviour of a single dowel-type fastener connection under shear loads. The fastener is modelled as elasto-plastic steel beam elements and the wood embedment is modelled as continuous nonlinear compression-only springs along the fastener length. The load-displacement relationship $p(w)$ of the nonlinear springs is shown in Figure 2. Eq. (1) gives the force-displacement backbone and Eq. (2) gives the reloading stiffness degradation of the embedment. A total of 6 parameters (Q_0, Q_1, Q_2, D_{max}, K_0, and α) need to be calibrated for the embedment properties. The HYST algorithm is able to capture the strength/stiffness degradation and pinching effect in dowel-type connections. Given the displacement history Δ, the shear load F transferred by the metal fastener between two layers of materials can be calculated quickly with normal computers. Although the HYST model represents a single-fastener connection, it can also be used to model the behavior of multi-fastener connections, such as a hold-down connection because a multi-fastener connection's hysteretic behavior is similar to that of a single-fastener connection. Of course, the size of the single fastener need to be increased and the embedment properties also need to be

calibrated to match the HYST calculations with a multi-fastener connection.

In this study, to reduce the number of parameters requiring calibration, the top layer in HYST was simply assumed as a 10 mm thick steel plate. The bottom layer was modelled as a wood layer with the six parameters calibrated by a random search approach. Detailed introduction of the HYST algorithm, the calibration process and the model validations and applications in connections and timber-framed shear walls can be found in literature (Foschi, 2000; Li, et al. 2012).
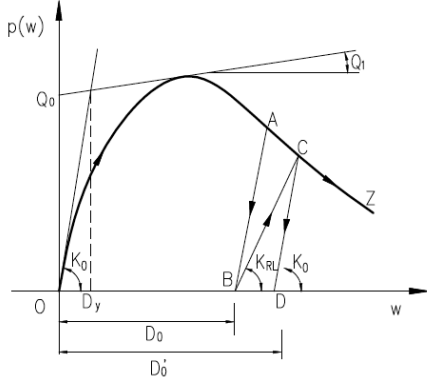
Figure 2 Force-deformation properties for embedment springs

$$\begin{cases} p(w) = (Q_0 + Q_1 w)\left(1 - e^{-K_0 w/Q_0}\right) & if \ w \leq D_{max} \\ p(w) = P_{max} e^{Q_3(w - D_{max})^2} & if \ w > D_{max} \end{cases} \tag{1}$$

where $P_{max} = (Q_0 + Q_1 D_{max})\left(1 - e^{-K_0 D_{max}/Q_0}\right)$ and $Q_3 = \frac{log(0.8)}{[(Q_2 - 1.0)D_{max}]^2}$.

$$\begin{cases} K_{RL} = K_0 & if \ D_0 < D_y \\ K_{RL} = \left(\frac{D_y}{D_0}\right)^\alpha K_0 & if \ D_0 \geq D_y \end{cases} \tag{2}$$

Where $D_y = Q_0/(K_0 - Q_1)$, the approximate yielding displacement of the embedment spring.

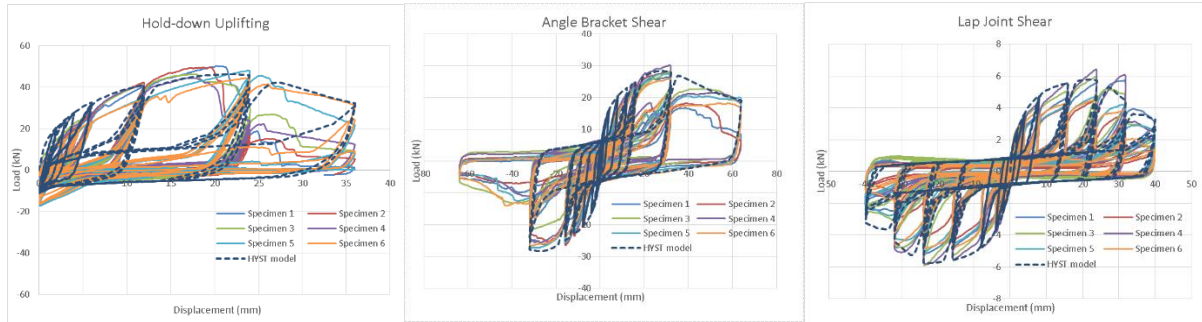## 3. HYST PARAMETER CALIBRATION FOR CONNECTIONS

In this study, the HYST parameters were calibrated by an experimental project conducted by IVALSA-CNR, Italy which covered Simpson HTT22 hold-down, BMF90x48x3x116 angle brackets, and HBS8x100 fully threaded self-tapping screws for panel-panel lap joints (Gavric, et al. 2012). Figure 3 shows the connection test setups. Each type of connections was tested in the vertical and horizontal directions following European standard EN12512 on timber connections. For the vertical tests of hold-downs and angle brackets, the load-deformation characteristics under uplifting are of main interest because the compression forces are resisted by the contacts between the CLT panel and the supporting member. For HYST model calibration, the average result of 6 connection replicates was used. Table 1 lists the calibrated HYST parameters and Figure 4 shows that the calibrated HYST agreed reasonably well with test results.
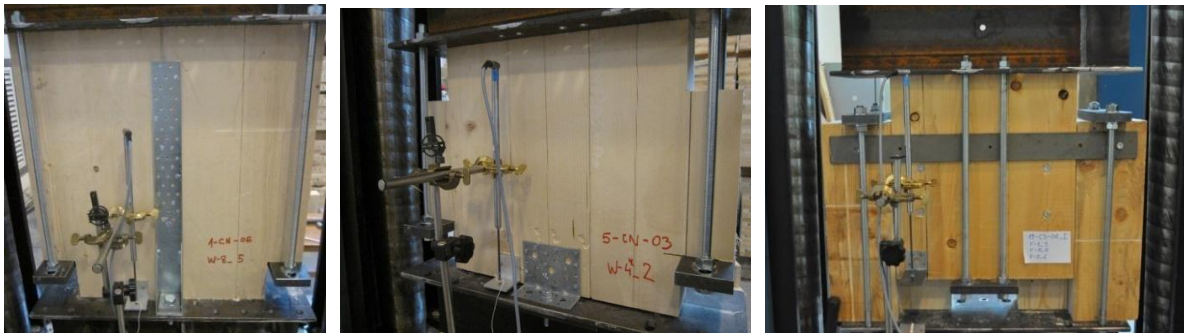
Figure 3 Test photos of hold-down, angle bracket and lap joint (Gavric, 2012)

Table 1 HYST parameters calibrated by connection test data

HYST parameters		Hold-down uplift	Hold-down shear	Angle-bracket uplift	Angle-bracket shear	Lap Joint shear	Lap Joint separation
Equivalent	*Length (mm)*	150	150	150	150	100	100
Fastener	*Diameter (mm)*	5	5	4.8	4.8	3.25	3.25
	$Q_0(kN/mm)$	8.0	0.26	2.8	3.4	2.0	2.0
Embedment	$Q_1(kN/mm^2)$	0.15	0.03	0.08	0.1	0.001	0.001
Properties	Q_2	1.35	1.2	1.4	1.5	1.25	1.25
P(w)	$K_0(kN/mm^2)$	5.0	0.8	1.25	1.5	0.6	0.6
	$D_{max}(mm)$	16	45	14	20	20	17
	α	0.2	0.2	0.2	0.2	0.2	0.2

Figure 4 Calibrated HYST loops vs Connection test results

4. CLTWALL2D VALIDATION

A series of CLT shear walls have also been tested in the SOFIE project led by IVALSA, Italy (Gavric, 2012). Test results of six walls were used to validate the CLTWALL2D model. Three of the walls were single-panel walls and the remaining three were coupled walls with two smaller panels, as shown in Fig. 5. These walls were 2.95m long and 2.95 m tall and consisted of 5-layer 85 mm thick panels. Each wall had two HTT22 hold-downs and four angle brackets installed to prevent uplifting and sliding. In the coupled walls, additional lap joints with self-tapping screws were installed vertically to connect the panels. These hold-down, angle bracket and lap joints were identical to those tested in the connection tests. Therefore, the calibrated HYST model parameters in Table 1 were readily embedded into CLTWALL2D. Table 2 lists the configurations of the six walls. Wall 1.2, Wall1.3 and Wall1.4 were single-panels walls with the same configurations except that the gravity loads on Wall 1.3 was reduced to a half. Wall2.1, Wall2.2 and Wall2.3 were coupled walls with the same gravity loads applied on the top of the wall. Wall2.1 and Wall2.2 were identical with lap joints spaced at 150 mm while Wall2.3 had half of the lap joints.

**Table 2 Configurations of tested shear walls**

Wall ID	No. of panels	Hold-downs (x2)	Angle-bracket (x4)	Vertical joints	Dead load
1.2 1.4	Single	HTT22	BMF90x48x3x116	N/A	18.5kN/m
1.3		HTT22	BMF90x48x3x116	N/A	9.25kN/m
2.1 2.2	Double	HTT22	BMF90x48x3x116	HBS8x100 @150mm	18.5kN/m
2.3		HTT22	BMF90x48x3x116	HBS8x100 @300mm	18.5kN/m

The stiffness properties of the CLT panels were calculated based on the equivalent stiffness of multi-layer orthotropic materials following CLT handbook (FPInnovations, 2011). For 5-layered 85 mm CLT panels, the calculated equivalent E_x was 4.6 GPa in the horizontal direction E_y was 6.7 GPa in the vertical direction assuming the wood boards used to manufacture the CLT panels had an average E_0 of 11 GPa and E_{90} of 0.35GPa. The shear-through-thickness rigidity G_{xy} was assumed to be 1.0 GPa. Poisson ratios

γ_{xy} and γ_{yx} were assumed to be 0.19 and 0.27, respectively.

The same displacement loading protocol used in the experimental studies was applied on the CLTWALL2D models. Figure 6 shows the model predicted load-drift hysteresis compared with the test results of the six walls. The model predictions agreed reasonably well with the test results in terms of peak loads and initial stiffness. Overall, the model slightly underestimated the total energy dissipation because the frictional effect between the CLT panels has not been considered in the model.
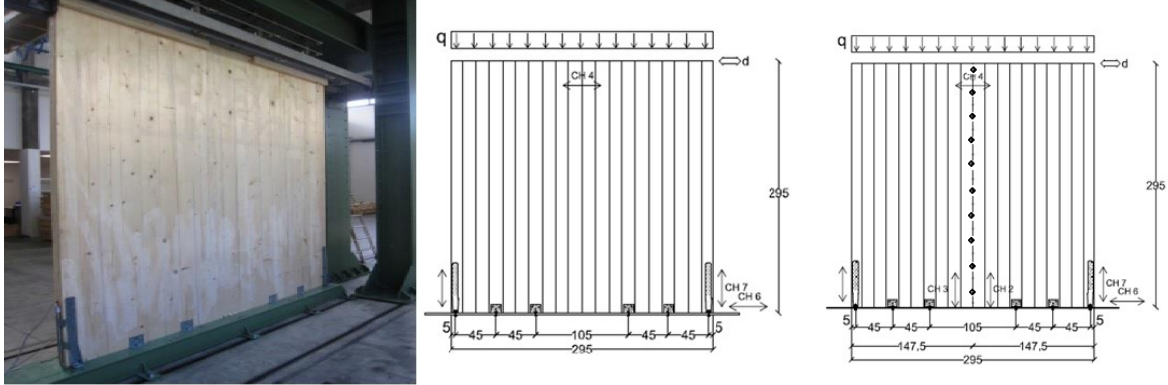

Figure 5 Experimental tests on single-panel and coupled CLT walls (Gavric, 2012)
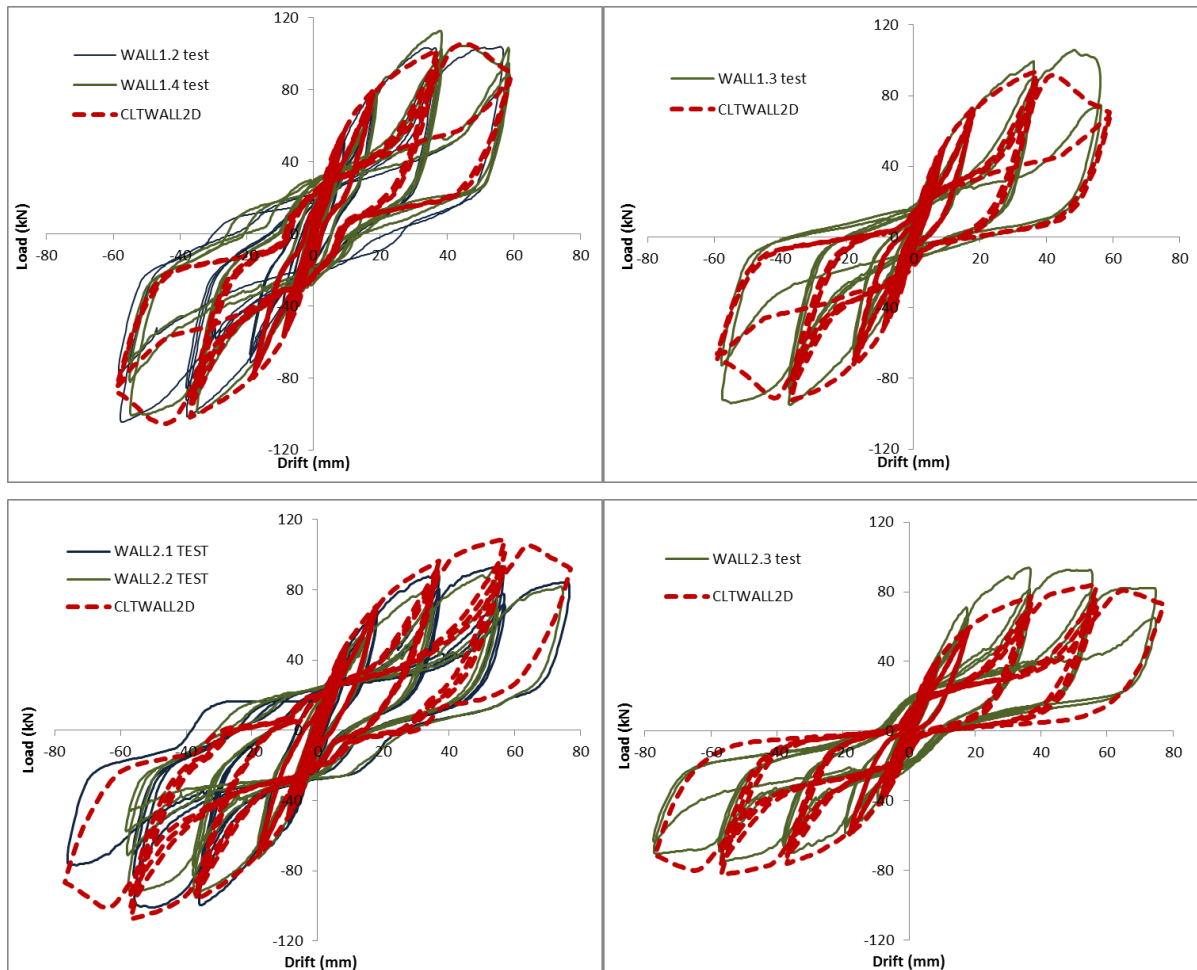
Figure 6 CLTWALL2D predictions vs experimental results

5. PARAMETRIC STUDIES

Parametric studies by CLTWALL2D were carried out to study the influence of gravity loads on the behavior of the single single-panel wall and the coupled wall with vertical lap joints spaced at 300mm. Four levels of gravity loads, 0kN/m, 9.25 kN/m, 18.5 kN/m and 27.75 kN/m, were used. Figure 7 shows the load-drift hysteresis of the walls subjected to the same displacement protocol. Table 3 lists the wall characteristics in terms of strength, initial stiffness, ductility and energy dissipation following the approach to calculate the equivalent energy elastic-plastic curve provided in ASTM E2126 (ASTM, 2010).
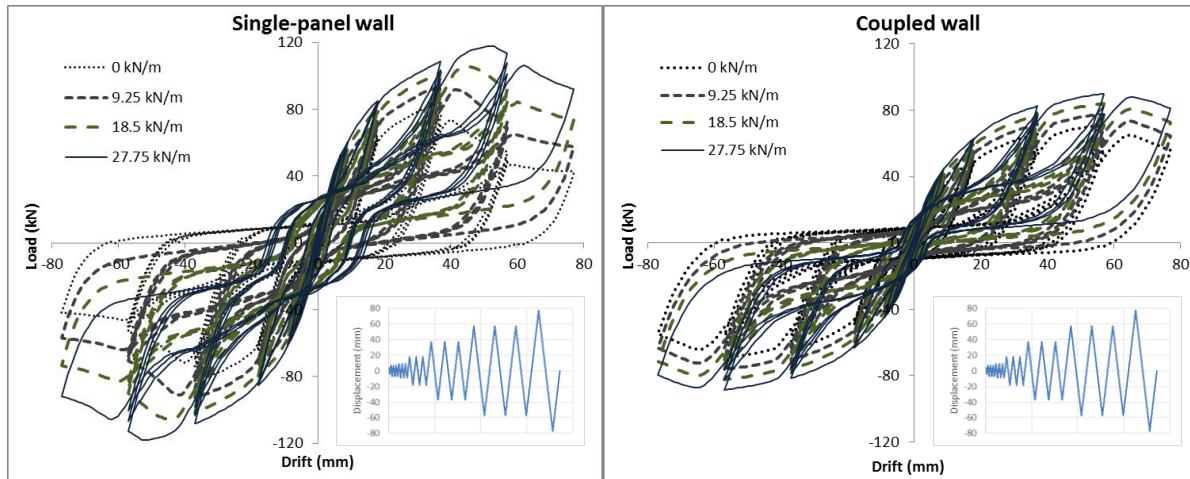

Figure 7 Influence of gravity loads on single-panel walls and coupled walls with lap joints @ 300mm

The simulation results indicated the significant influence of gravity loads on the wall behavior. For the single-panel wall, the gravity loads with an increment of 9.25 kN/m led to a 5% to 15% increase in ultimate capacity, initial stiffness, ductility ratios and total energy dissipation. For the coupled wall, such a gravity load increase led to a 7% to 14% increase in ultimate capacity and initial stiffness. However, the ductility ratios and energy dissipation increase was about 1% to 4%.

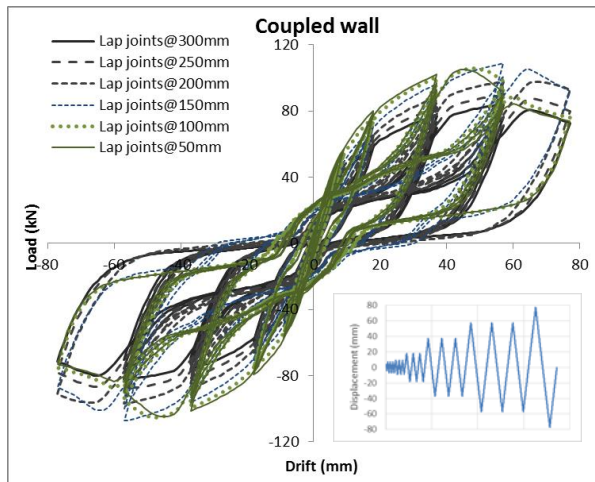**Table 3 Summary of wall characteristic with different gravity load levels**

	Single-Panel walls with Gravity load				Coupled walls (lap joints @ 300 mm) with Gravity load			
	0 kN/m	9.25 kN/m	18.5 kN/m	27.25 kN/m	0 kN/m	9.25 kN/m	18.5 kN/m	27.25 kN/m
P_{max} (kN)	84.5	97.0	109.2	120.1	72.7	79.1	85.6	92.0
D_{max} (mm)	37	41	47	54	56	58	60	62
F_y (kN)	73.4	83.9	94.0	103.6	64.3	70.2	76.0	81.6
D_y (mm)	11.9	11.9	12.2	12.7	14.9	14.3	14.0	13.8
μ	4.2	4.9	5.5	6.3	5.4	5.6	5.7	5.8
K (kN/mm)	6.2	7.1	7.7	8.2	4.3	4.9	5.4	5.9
Tot. Enrg (kJ)	24.9	27.4	28.6	30.0	20.2	21.6	21.7	21.9

Further parametric studies were also carried out to study the influence of vertical joint density on the behavior of the coupled wall. The spacing of the vertical lap joints were selected to be in a range of 50 mm to 300 mm with an increment of 50 mm. As expected, when the vertical lap joints are sufficient, e.g. spaced at 50mm, the coupled wall will be performing closely to a single-panel wall, as shown in Fig. 8. With lap joints increased to 300mm, the coupling effect becomes much weaker.

Coupled wall with lap joints @50mm   Coupled wall with lap joints @150mm   Coupled wall with lap joints @300mm
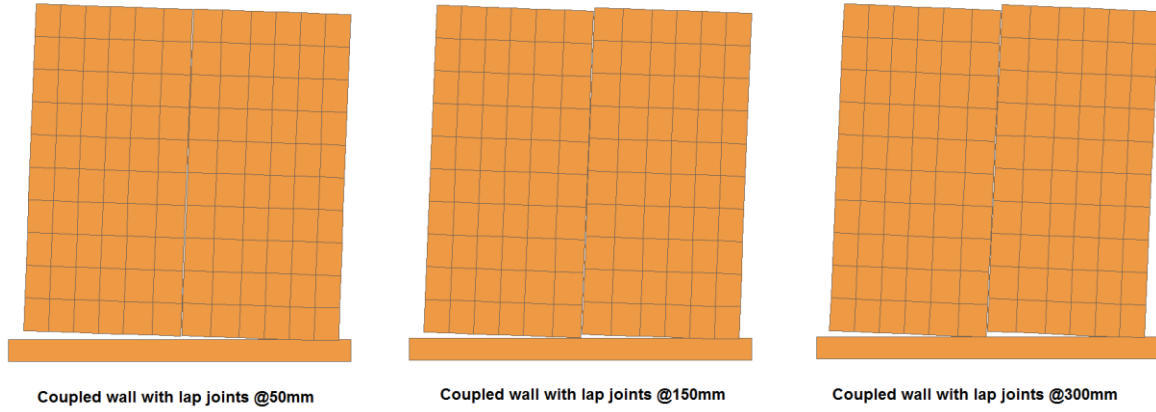
Figure 8 Coupled wall deformations at a drift of 50 mm

Figure 9 shows the hysteresis of all six walls with varying lap joint spacing from 50 mm to 300 mm. Table 4 shows the summary of the wall characteristics. Interestingly, when the spacing increased from 50 mm to 150 mm, the wall capacity and total energy dissipation did not change significantly, but the initial stiffness decreased significantly. Further increase the spacing from 150mm to 300 mm with an increment of 50mm led to significant decrease in the wall capacity, initial stiffness and total energy dissipation. However, the ductility ratio increased slightly.

Figure 9 Influence of lap joint density on coupled walls

Table 4 Summary of wall characteristic with different lap joint spacing

	Coupled walls with lap joints spaced @					
	50mm	**100mm**	**150mm**	**200mm**	**250mm**	**300mm**
P_{max} (kN)	109.1	109.5	110.0	100.3	91.6	85.6
D_{max} (mm)	47	50	59	65	62	60
F_y (kN)	94.0	94.2	94.7	88.9	81.3	76.0
D_y (mm)	12.0	13.5	15.7	15.5	14.6	14.0
μ	5.6	5.3	5.1	5.2	5.5	5.7
K (kN/mm)	7.8	7.0	6.0	5.7	5.6	5.4
Tot. Energy (kJ)	29.3	29.5	29.3	26.3	23.7	21.8

6. Conclusions

This paper first introduced a finite element shear wall model called CLTWALL2D to simulate the lateral behavior of CLT shear walls under reversed cyclic loads. Model input parameters of critical connections such as hold-downs, angle brackets, and lap joints were calibrated based on a connection test database. The wall model was validated against the test results of six walls and reasonably good agreement was

observed between the model predictions and the test results. Based on the experimental results and parametric studies by CLTWALL2D simulations, the following conclusions were drawn:

1) Gravity loads affect the shear strength and stiffness of CLT walls significantly. Walls carrying higher gravity loads tend to have higher strength, stiffness, ductility and energy dissipation under the same cyclic loading protocol. This is because the gravity load can counter-act the rocking motion under static and reverse cyclic loading.

2) Coupled walls do not necessarily mean higher ductility or more energy dissipation although they use more connections. The parametric study indicated that when the number of vertical lap joints decreases, the strength and stiffness of the wall decreases while the ductility level does not change significantly. Therefore, in order to achieve the optimal behavior of the coupled CLT walls, the number and capacity of hold-downs, angle brackets and vertical joints should be carefully selected.

3) To eliminate high experimental cost, the CLTWALL2D model provides a useful tool to study the lateral behavior of CLT shear walls of different configurations as along as the behavior of the connections can be well calibrated by experimental studies or other trustworthy data sources.

Acknowledgement

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