WOOD RESEARCH
64 (2): 2019
213-222

EVALUATION OF HORIZONTAL SHEAR PERFORMANCE OF LARCH CLT WALLS ACCORDING TO THE EDGE CONNECTION SHAPE

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(Received June 2018)

ABSTRACT

In this study, cyclic tests were performed on the larch CLT shear walls depending on the half lap reinforcement of half-lap connections and reinforced plywood of spline connections in order to evaluate the horizontal shear performance of the larch CLT walls. The test results show that there is no difference in residual strength depending on the reinforcement of half-lap connection, but their initial stiffness has been increased by 9%. There was no significant difference either in the residual strength of double spline connections depending on the application of reinforced plywood, while the spline reinforcement has failed to increase initial stiffness. All of the larch CLT walls constructed according to the edge connection shape were measured to have a strength reduction ratio of less than 10% in each horizontal displacement intervals and an equivalent viscous damping ratio of less than 10% for energy dissipation in the initial and final horizontal displacement intervals, thereby confirming that their excellent horizontal shear performance and seismic performance.

KEYWORDS: *Larix kaempferi* Carr., cross-laminated timber, horizontal shear performance, seismic performance, half-lapped, spline.

INTRODUCTION

Given that construction, expansion or refurbishment of concrete buildings are required for general urban regeneration projects in South Korea with over 90% urbanization rate, it is possible to contribute to eco-friendly city design by introducing wooden architectural methods for cityscape. Urban wooden architecture will help us build an environmentally friendly city by not only reducing but also stabilizing inevitable carbon emissions inherent in urban development and growth. In this respect, the cross-laminated timber (CLT), which can be used as a wall, floor and roof with a large interval by cross-laminating three or more layers with an adhesive or via mechanical manner, has a great potential for booming up urban wooden architecture (Sadeghi et al. 2015, Dagenais 2016, Song 2018, Popovski and Karacabeyli 2012). The CLT architecture can compete with other brick-and-concrete-based construction systems thanks to its ease of handling and high level of prefabrication during construction of middle- and high-rise buildings (Bogensperger et al. 2011, Mohammad et al. 2012). Comprehensive studies that have attempted to quantify the seismic behavior of low- and middle-rise CLT structure buildings, including Italy's SOFIE project, have proven that the panel-panel connections such as half-lap and spline as well as the joints used for wall-foundation such as metal brackets and fasteners among the components of CLT can reduce the rigidity of high walls while increasing their ductility and energy dissipation (Ceccotti et al. 2006, Lauriola and Sandhaas 2006, Ceccotti and Follesa 2006, Ceccotti et al. 2013). However, the existing studies on the horizontal strength performance of the CLT wall have mainly focused on single walls, and even the studies on multi-panel walls were limited to the walls connected by half-lap method. In this study, therefore, a horizontal shear strength performance test was carried out on the larch CLT walls that are side connected with half-lapped connection and with other connections with their half-lap reinforced with GFRP sheets, as well as on other larch CLT walls that are side connected with double-spline connections using plywood and GFRP reinforced plywood.

MATERIALS AND METHODS

Production of CLT shear walls

In this study, 5-layer CLT (thickness: 130 mm) were fabricated with larch (*Larix kaempferi* Carr.) laminae for the test. The average air-dried moisture content of lamina was 11% and the average air-dried gravity was 0.53. Phenol-resorcinol formaldehyde adhesive was used to bond together the laminae. The aspect ratio of the CLT wall was 1: 0.6, while the height was 1800 mm and the width was 1100 mm. The half-lap and double-spline connections were applied to the edge connections of the CLT walls before they were fabricated as shown in Tab. 1 depending on whether they were reinforced by glass fiber reinforced plastic(GFRP) or according to the diameter and length of the self-tapping screw (STS). Series-HN-8-120 is a shear wall connected to a non-reinforced half-lap connection using an STS measuring 8 mm (d) × 120 mm (l), while Series-HR-8-120 is a wall connected to a half-lap connection reinforced with GFRP using an STS measuring 8 mm (d) × 120 mm (l).

Tab. 1: Horizontal shear test schedule.

Series	CLT panel sizes(*mm*)	Type of edge connection	Reinforcement of half-lap(*mm*)	Reinforcement of spline(*mm*)	Screw (*mm*)
Series-HN-8-120	1800×1000	Half-lap	Non-reinforced	-	8×120
Series-HR-8-120	1800×1000	Half-lap	Reinforced with GFRP sheet 2.6 (t) × 80 (w)	-	8×120
Series-DP-6-70	1800×1000	Double-spline	-	Larch plywood 24 (t) × 120 (w)	6×70
Series-DR-6-70	1800×1000	Double-spline	-	Reinforced plywood with GFRP 24 (t) × 120 (w)	6×70

The edge distance was set as 5D for the design of half-lap and double-spline connections while fastener spacing was set as 21.75D by referring to the evaluation report 13677-R of Canadian Construction Material Center (CCMC 2013). Series-DP-6-120 is a wall connected to a plywood double-spline connection which uses an STS measuring 6 mm (d) × 70 mm (l), while Series-DR-6-70 is are inforced plywood double-spline connection using an STS of the same specification as Series-DP-6-120. The fastener spacing was also determined as 21.75D as in the case of the wall with half-lap connection by referring to the CCMC (Fig. 1). Alarch plywood (thickness: 24mm) and GFRP reinforced plywood (GFRP thickness: 6mm, plywood thickness: 18mm) were each used as spline. And STS measuring 8 mm (d) × 120 mm (l) was used for the half-lap connection while an STS measuring 6 mm (d) × 70 mm (l) was used for the double-spline connection. The average air-dry density of plywood and reinforced plywood were 553.4 kg·m^{-3} and 901.1 kg·m^{-3}, respectively.

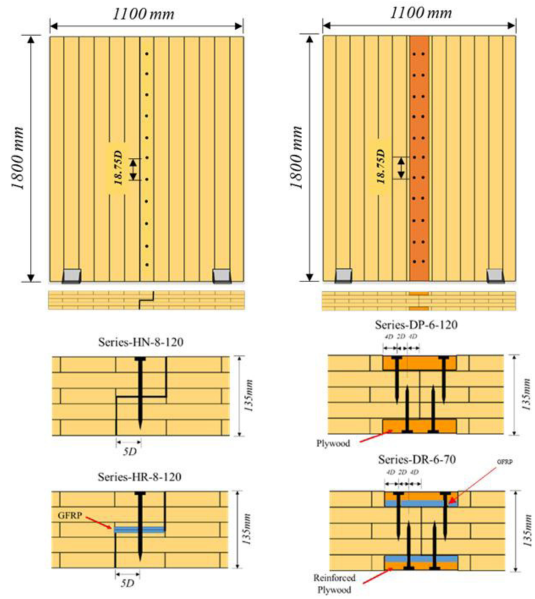

Fig. 1: Schematic diagrams of Series-HN-8-120 and Series-HR-8-120.

Test set-up

Fig. 2 shows a schematic diagram of the horizontal shear test of the CLT wall. The prefabricated metal brackets were used to join the CLT wall with the metal I beam base. The brackets were jointed to the front and rear surfaces, one at 90 mm from the center of the wall.

High tensile bolts measuring 12mm in diameter were used for the metal I beam while 14 STS's each measuring 3.2 mm (d) x 64 mm (l) were used to connect the bracket to the CLT wall. The installed wall was assumed as the first-floor wall of a two-story building before a vertical load of 10 kN·m^{-1} was equi distributed to the vertical load beam near the top of the wall, while a horizontal loading device coupled with the load cell was connected to the wall (Popovski et al. 2010). The crosshead that applies the positive load to the CLT and the stopper that applies the negative load experienced an increased frictional resistance as they are angled at the contact surface with the specimen. Therefore, a spring bearing was installed between the test specimens to minimize the frictional resistance. A vertical load beam and I beam in the base were fastened together with a tie rope at a point some 550 mm from the center of the wall all while the vertical load was maintained. Meanwhile, the behavior of the timber wall with respect to the horizontal load is expressed as three cases as shown below (Dujic et al. 2006).

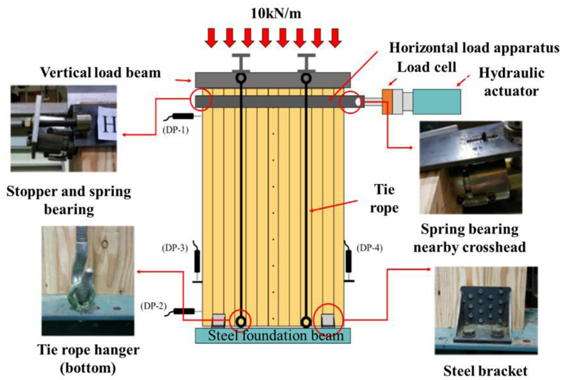

Fig. 2: Schematic diagrams of horizontal shear test set-up for CLT wall.

Case 1: Shear cantilever mechanism, where one edge of the panel is supported by the firm base while the others can freely translate and rotate.

Case 2: Restricted rocking mechanism, where one edge of the panel is supported by the firm base while the others can translate and rotate as much as allowed by the ballast that can translate only vertically without rotation.

Case 3: Shear wall mechanism, where one edge of the panel is supported by the firm base while the other can translate only in parallel with the lower edge and rotation is fully constrained.

The tie rope installed in this test prevents the wall from engaging in rocking behavior and then failing at the foundation as shown in Case 1. It also triggers an occurrence of shear stress in the connection between the CLT panels as shown in Case 2, thereby allowing us to observe the behavior depending on the type of connection.

Test procedure

As shown in Fig. 2, the displacement of the CLT wall with respect to the horizontal load was measured by placing displacement transducers with maximum measuring capacity of 50mm (CDP-50) at one point 200 mm from the opposite upper side of the load cell (DP-1) and another point some 100 mm from the bottom (DP2). The displacement with respect to the vertical load was measured by installing displacement transducers at points (DP-3, DP-4) each 300 mm from the left and right sides of the wall base. The displacement of the displacement transducer according to the changing load was measured using a data logger (TDS-302).

With regard to the cyclic loadings schedule, the drift angles specified in AIJ (1/600 rad., 1/450 rad., 1/300 rad., 1/200 rad., 1/150 rad., 1/100 rad., 1/75 rad., 1/50 rad) were transformed into horizontal displacements using Eq. 1, and positive and negative loads were applied for over three time along each horizontal displacement interval. The loading speed was set at 40 mm.min^{-1} (AIJ 2012).

$$\Delta_{H.} = \gamma/\Delta_{DP} \tag{1}$$

where: $\Delta_{H.}$ = horizontal displacement (mm),
γ = drift angle (rad.),
Δ_{DP} = distance between DP-1 and DP-2 (mm).

RESULTS AND DISCUSSION

Analysis of hysteresis cycles

Strength

The residual strengths of the test specimens after up to the third cycle of ± 32.0 mm (1/50 rad.) were compared along the respective hysteresis loop curve of the test specimens. The results confirmed that there is no reinforcement effect of half-lap since the residual strengths of HN-8-120 and HR-8-120 were measured as 31.7 kN and 28.4 kN, respectively. The residual strengths of DP-6-70 and DR-6-70 were measured as 27.6 kN and 28.4 kN, respectively, thereby confirming that the type of spline did not affect the horizontal shear strength of the wall. Barely visible shear displacement was observed during the test at the edge connections between the panels of all CLT walls, but in general the CLT walls demonstrated the same behavior as the rigid body. In addition, the displacement at the connection between the CLT wall and the foundation contributed greatly to the displacement of the CLT wall. These results are in contrast to the study of Popovski and Karacabeyli (2012) which reported that displacement of the panel-to-panel edge connections contributes significantly to the total displacement of the CLT wall, the reason being that the loading protocol in this study was composed of relatively low horizontal displacement (or drift angle) as compared to those adopted by the preceding study of Popovski and Karacabeyli (2012).

Timber structures jointed with metal fasteners are sensitive to strength degradation at the joint under cyclic loadings. As a result, strength degradation is an important parameter to confirm the resistance to repeated seismic movements (Sheikhtabaghi 2015, Pozza et al. 2014). The strength degradation (ΔF) signifies a reduction of the load when it has a constant displacement (DIN 12512-2002). The strength degradation in this study was calculated as the difference between the maximum load in the first cycle and the maximum load in the third cycle within the same displacement interval (Eq. 2).

$$\Delta F = 1 - \left(\frac{F_3}{F_1}\right) \times 100 \tag{2}$$

where: ΔF - strength degradation (%),
F_1 - force at first cycle (kN),
F_3 - force at third cycle (kN).

As a result, there was no dramatic decrease in strength within the amplitude of particular cycle, while the entire walls demonstrated an excellent performance with less than 10% strength degradation. The hysteresis loops and back-bone curve are shown in Fig. 3.

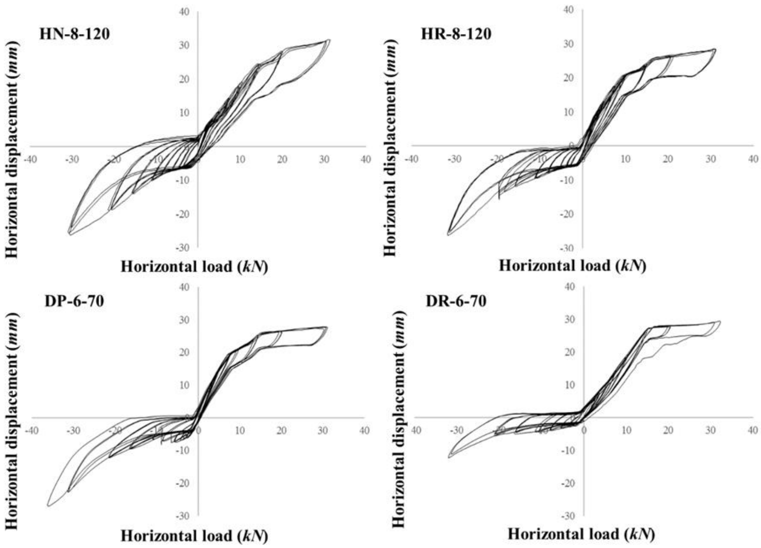

Fig. 3: Hysteresis loops of larch CLT shear walls.

Tab. 1: Strength degradation at each horizontal displacement.

Horizontal displacement (mm)	HN-8-120	HR-8-120	DP-6-70	DR-6-70
±2.7 (1/600 rad.)	0.6%	-3.7%	-2.9%	1.9%
±3.6 (1/450 rad.)	-1.5%	-5.0%	-4.2%	4.3%
±5.3 (1/300 rad.)	1.1%	0.6%	6.3%	-6.3%
±8.0 (1/200 rad.)	-0.5%	0.6%	1.3%	-0.6%
±10.7(1/rad.)	3.3%	4.6%	1.7%	3.1%
±16.0(1/rad.)	2.8%	1.2%	0.9%	1.2%
±21.3 (1/75 rad.)	1.4%	1.0%	0.3%	-0.3%
±32.0 (1/50 rad.)	1.4%	-1.2%	0.2%	0.1%

Stiffness

Fig. 4 show the back-bone curves drawn by connecting the dots of maximum loads representing the cycle on the hysteresis loop as shown in the left figure. The initial stiffness was determined by the slope of the initial straight-line of the back-bone curve. The initial stiffness of the HR-8-120 with a half-lap connection reinforced with GFRP was 9% higher than that of unreinforced HN-8-120, thereby confirming that the half-lap reinforcement has an effect of increasing the initial stiffness rather than increasing the strength. On the other hand, the initial stiffness of DP-6-70 was 3.3 kN·mm^{-1}, and the initial stiffness of DR-6-70 was 2.1 kN·mm^{-1}, the reason being that fasteners with a diameter of 6 mm are easily deformed by the highly density reinforcing spline.

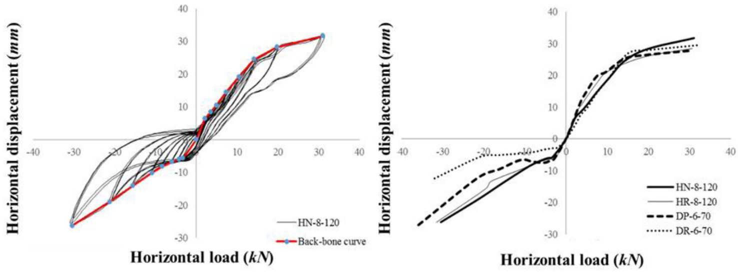

Fig. 4: Hysteresis loops and back-bone curve.

The stiffness degradation according to the modified intervalwas calculated using Eq. 3.

$$\Delta K = 1 - (K_{Amplitude,n}/K_{Amplitude,2.8}) \times 100 \quad (3)$$

where: ΔK - stiffness degradation (%),
$K_{Amplitude,n}$ - stiffness at certain horizontal displacement (kN·mm^{-1}),
$K_{Amplitude,2.8}$ - stiffness at ±2.8 (kN·mm^{-1}).

Tab. 2 shows the stiffness and stiffness degradation according to the horizontal displacement interval. The stiffness of the half-lapped specimens decreased by 50% and 44% in the horizontal displacement of + 3.6 mm (1/600 rad.), respectively, whereas that of the spline test specimens tended to decrease or even increase to 3% and -14%. For the HN-8-120, the stiffness degradation was kept at50% from + 3.6 mm (1/600 rad.) to + 16.0 mm (1/100 rad.) before jumping back to 90% degradation compared to the initial stiffness in the last + 32.0 mm (1/50 rad.).The stiffness of the HR-8-120 decreased sharply beginning from + 16 mm (1/100 rad.), falling significantly in the earlier horizontal displacement than the HN-8-120, thereby confirming that the half-lap reinforcement is effective in increasing the initial stiffness but is ineffective under cyclic loading. DP-6-70 showed virtually no decrease in stiffness in the + 3.6 mm (1/600 rad.) unlike the previous two half-lapped shear walls but its stiffness decreased by 82% in the + 10.7 mm (1/150 rad.). And the stiffness degradation was measured as 97% in the last horizontal displacement. In the case of DR-6-70, the initial stiffness was measured to be relatively lower than that of the other test specimens but showed a low stiffness degradation despite an increase in horizontal displacement. The stiffness was found to be decreasing sharply in + 21.3 mm (1/75 rad.).

Tab. 2: Stiffness and stiffness degradation of CLT panels.

Horizontal displacement (mm)	Stiffness (kN·mm^{-1})			
	HN-8-120	HR-8-120	DP-6-70	DR-6-70
	Forward	Forward	Forward	Forward
±2.7 (1/600 rad.)	3.0	3.2	3.3	2.1
±3.6 (1/450 rad.)	1.5 (50%)*	1.8 (44%)	3.2 (3%)	2.4 (-14%)
±5.3 (1/300 rad.)	1.5 (50%)	2.0 (37%)	2.2 (33%)	1.4 (33%)
±8.0 (1/200 rad.)	1.7 (43%)	1.8 (44%)	1.9 (42%)	2.0 (5%)
±10.7 (1/150 rad.)	1.5 (50%)	1.4 (56%)	0.6 (82%)	1.6 (34%)
±16.0 (1/100 rad.)	1.4 (53%)	0.6 (81%)	0.9 (73%)	1.7 (19%)
±21.3 (1/75 rad.)	0.7 (77%)	0.4 (87%)	0.2 (94%)	0.2 (90%)
±32.0 (1/50 rad.)	0.3 (90%)	0.2 (94%)	0.1 (97%)	0.1 (95%)

( )*Stiffness degradation

Energy dissipation

Tab. 3 shows equivalent viscosity damping ratios in the primary and tertiary cycles of ± 5.3 (1/300 rad.) which is estimated as proportional limit area along the hysteresis loop and ± 34.0 (1/50 rad.) which is the last horizontal displacement interval.

Tab. 3: Equivalent viscous damping ratio (v_eq).

Horizontal displacement (mm)	HN-8-120		HR-8-120		DP-6-70		DR-6-70	
	1st	3rd	1st	3rd	1st	3rd	1st	3nd
±5.3 (1/300 rad.)	8%	7%	7%	8%	5%	6%	9%	10%
±32.0 (1/50 rad.)	8%	7%	7%	7%	3%	5%	5%	6%

Even though all test specimens were in the last horizontal displacement interval, the equivalent viscosity damping ratio did not exceed by 10% on average in both the primary and tertiary cycles. It was concluded based on these results that the CLT walls in this study have good seismic performance against cyclic loading.

CONCLUSIONS

In this study, cyclic tests were performed on the horizontal shear strength of the walls that were constructed according to the types of edge connection between CLT walls.

The half-lap reinforcement increased the initial stiffness of the CLT wall but did not increase their strength. Furthermore, it failed to increase the stiffness under cyclic loading either. The CLT wall with reinforced spline was measured to have a low horizontal strength as the fasteners with small diameter deformed easily by the spline with an excessively high density, Therefore, the appropriate diameter of the fastener according to the density of the spline will have a significant effect on the improvement of the horizontal strength performance of CLT shear wall.

The difference in horizontal shear strength between the walls with half-lapped and double-spline connections was not great. In conclusion, it is judged to be suitable for building structures designed to have seismic resistance as it has a damage ratio and equivalent viscous damping ratio of less than 10%. However, it was confirmed in this study that the larch CLT wall connected to the half-lap connection has a better workability than the larch CLT shear walls with double-spline connection, and that it can reduce the number of required fasteners and does not require the use of splines.

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