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EXPERIMENTAL TESTING OF HIGH-CAPACITY SINGLE AND COUPLED CLT SHEAR WALL SYSTEMS

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ABSTRACT: This paper presents an experimental study on three types of balloon-framed cross-laminated timber (CLT) shear wall configurations: (a) a single wall, (b) a coupled wall with a half-lap joint between wall piers, and (c) a coupled wall with steel link beams between wall piers. Three-storey, 2/3-scale CLT wall specimens were cyclically tested to failure and the results are presented to compare the performance between wall types. All specimens used 5-ply, 205mm-thick CLT wall panels with the same base connections of mixed-angle screw hold-downs and notched shear keys. The beam-coupled wall achieved a peak strength of 592kN and demonstrated the best performance among the wall configurations with the highest strength, stiffness, energy dissipation, and least amount of degradation under repeated load cycles. All the wall specimen exhibited ductile failure modes and demonstrated their feasibility to be designed for lateral load resisting systems in buildings.

KEYWORDS: CLT, Shear Walls, Cyclic Loads, Self-tapping Screws, Coupled Walls, Hold-downs

1 INTRODUCTION

Cross-laminated timber (CLT) buildings are gaining global interest from architects, engineers, and owners. For CLT buildings built in seismic regions, CLT shear walls are critical to providing adequate earthquake resistance. It is well recognised that the lateral performance of typical CLT shear walls is governed by their connections [1], primarily because: (a) CLT panels have inherently high in-plane strength and stiffness and (b) conventional connections used in CLT structures have relatively low strength and stiffness.

Pei et al. [2] presented a review of experimental research on conventional CLT shear walls for seismic resistance which typically follow platform construction methods and use commercially available light gauge metal hold-downs or shear brackets with small diameter nails or screws. Connections with small, slender fasteners can achieve ductile behaviour and dissipate energy by fastener yielding and local timber embedment crushing around the fasteners [3]. They are currently the industry standard for connecting CLT walls as they are readily available and easy to specify. However, these connectors normally have a limited design capacity of less than 100 kN [4] and are often not strong and stiff enough to impose significant in-plane stresses on the CLT wall panels, particularly when relatively thick panels with 5 or 7-ply layups are used. Therefore, conventional connectors are not structurally efficient for applications in multi-storey buildings with CLT shear walls subjected to high seismic or wind demands.

Compared with small diameter nails and screws, larger diameter steel dowels or bolts can be used to create higher capacity hold-downs in CLT shear walls. Large-scale hold-down testing on 5-ply CLT panels with a group of 16-Ø20mm mild steel dowels and 25mm-thick steel knife plates found maximum connection strength exceeded 1000kN [5]. Further experimental research [6] showed the advantages of increased row spacing and end distance to encourage more ductile hold-down responses under cyclic loading.

Self-tapping screws have been widely used in mass timber construction due to their relatively high strength and ease of installation. When installed at 90° to the timber grain, slender screws can provide ductile responses and behave like typical dowel-type fasteners. When installed at an inclined angle to the timber grain, these screws are engaged primarily in withdrawal and can provide high connection strength and stiffness. For mixed angle screw connections, it is possible to achieve a balanced behaviour with high strength, high stiffness, and high ductility. Wright et al. [7,8] tested mixed angle screwed hold-downs and found peak strengths exceeded 600kN.

The previously mentioned experimental programmes on dowelled and screwed hold-downs demonstrate the ability of CLT hold-downs to achieve higher strengths than currently available commercial connectors. Furthermore, shear keys at the base of walls (similar to castellated connections studied by Brown et al. [9]) allow much higher shear capacities when compared to conventional angle brackets with groups of ring shank nails or small diameter screws. The use of these higher capacity

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connections in combination with coupled wall configurations allows CLT shear walls to be a more practical solution in mid-rise buildings.

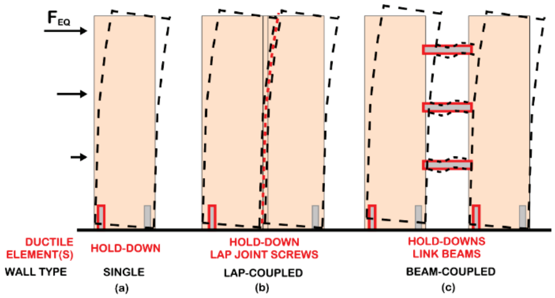

Figure 1: *(a) Single wall, (b) coupled wall with a lap joint, and (c) coupled wall with steel link beams.*

Figure 1 describes three types of multi-storey CLT walls: (a) a single wall, (b) a coupled wall with a screwed vertical half-lap joint (lap-coupled (LC)), and (c) a coupled wall with steel link beams (beam-coupled (BC)). The designated ductile elements where yielding is promoted in each wall type are also highlighted in Figure 1. All other components should be designed to have adequate strength to prevent premature failure in the system that may compromise its integrity during an earthquake. In this study, the hold-downs in each wall type are designed as ductile elements. In the LC and BC wall types, the lap joint screws and link beams, respectively, are additional ductile elements. The BC configuration is a form of hybrid wall system which uses ductile steel elements to enhance the hysteretic energy dissipation of a primarily timber-based structural system, similar in concept to the addition of steel buckling restrained braces in timber frames [10,11].

This study compares the lateral behaviour of the three different wall types with the results of four large-scale cyclic wall tests.

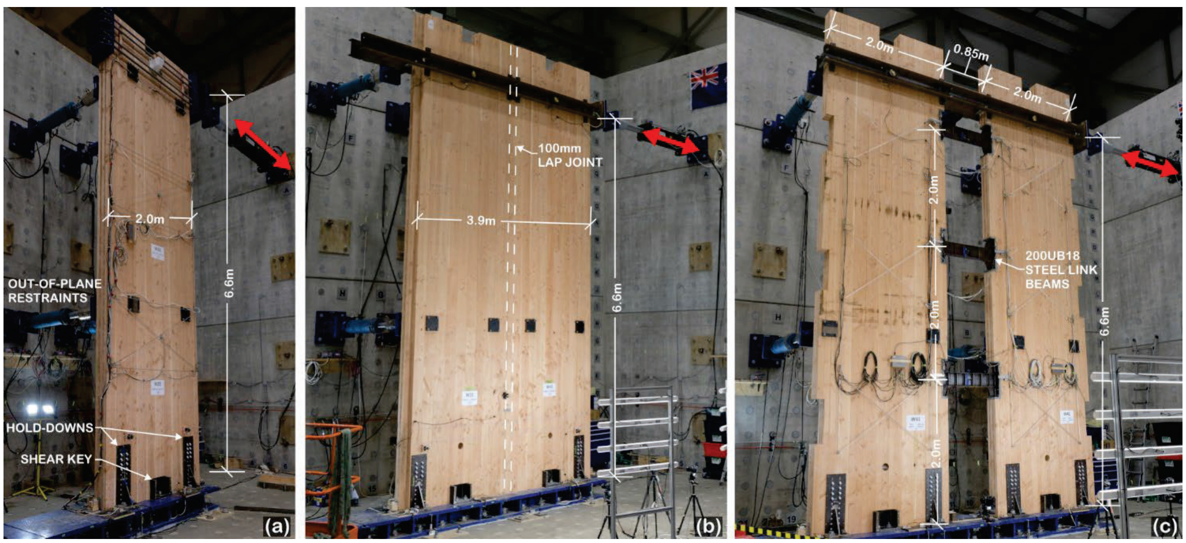

Figure 2: *Test specimen overview and experiment setup: (a) Single wall, (b) Lap-coupled wall, and (c) Beam-coupled wall.*

2 EXPERIMENT DETAILS

The test setup for each CLT wall configuration is shown in Figure 2. The specimens were proportioned and detailed as three-storey walls scaled by a factor of 2/3 and loaded laterally at a height of 6.6m. The CLT panels were created from grade SG8 (per NZS3603 [12]) Douglas-fir boards with a layup of 45/35/45/35/45mm.

Each wall specimen used the same type of mixed-angle screwed hold-downs, based on the previous testing by Wright et al. [7,8]. The ratio between the Ø12x260mm partially threaded inclined screws and the Ø12x180mm partially threaded inclined screws was 1:1.5. One hold-down bracket was installed on each side of the wall specimen. The combination of mixed angle screws creates a hold-down with high stiffness (due to the 45° screws) and reasonably high ductility (due to bending of the 90° screws). The typical hold-down and shear key used in each specimen are shown in Figure 3. The shear keys were installed in a full-depth, 250x350mm notch at the centre of the wall base. The vertical contact faces of the shear key were angled at a slope of 1:25 to prevent binding when the wall base rotated.

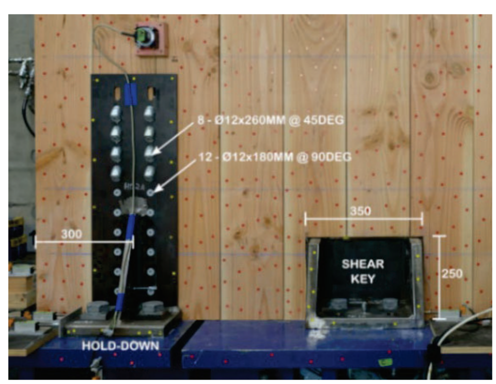

Figure 3: *Mixed-angle screwed hold-downs and shear keys.*

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The lap-coupled walls used Ø10x200mm partially threaded screws in the 100mm-wide vertical half-lap joint with spacings of 200mm and 100mm for wall specimens LC1 and LC2, respectively. Therefore, LC2 was designed to achieve a greater degree of coupling.

The beam-coupled wall (Figure 1c) used dissipative, 200UB18 steel link beams with capacity-designed [13] screwed end plate connections to provide coupling action between the CLT panels. Link beams were installed at each level with a vertical spacing of 2m.

Each specimen was subjected to the CUREE cyclic loading protocol [14], shown in Figure 4. The reference wall drifts to define the CUREE protocol were determined based on preliminary pushover analyses and are reported with the results in Table 1.

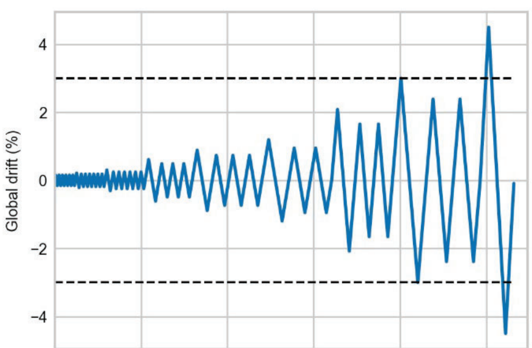

Figure 4: *Example of CUREE loading protocol (D_{ref}=3.0%).*

3 DAMAGE OBSERVATIONS

Each wall specimen primarily experienced damage in its designated ductile elements, which are described in Figure 1. No damage was observed at the shear keys in any test specimen. A moderate degree of toe crushing was experienced by each specimen due to a concentration of compression force but it did not lead to a brittle failure or result in sudden strength loss.

The mixed angle screwed hold-downs in each specimen experienced screw bending and withdrawal, in addition to local embedment crushing around the fasteners, as shown in Figure 5, due to the high uplift force demand. No screws in the hold-downs experienced fracture during the experiments, even though the maximum hold-down uplift displacement exceeded 100mm during the experiments.

The LC walls experienced significant damage to the screws in the vertical lap joint, as shown in Figure 6. Initial damage of screw bending and withdrawal was evident by the screw heads protruding from the timber surface (Figure 6a). Significant local crushing occurred internally around the lap-joint fasteners in the vertical direction (Figure 6b). Following completion of the test, the lap joint screws were uninstalled but many of them fractured due to the applied reverse torque (Figure 6c), indicating significant damage accumulated in the screws. Figure 6d shows the mechanism of a typical screw yielding in the half-lap joint.

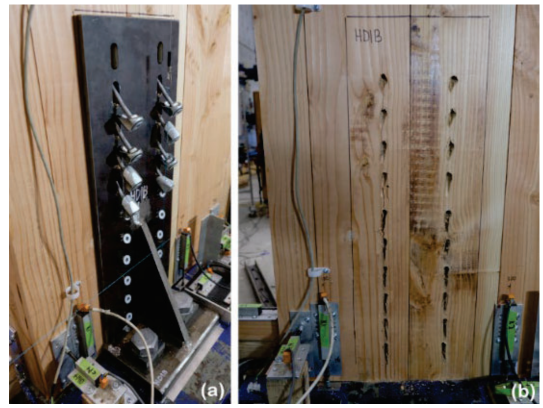

Figure 5: *Typical damage observed in mixed-angle screw hold-down connections: (a) screw withdrawal and (b) local wood crushing.*

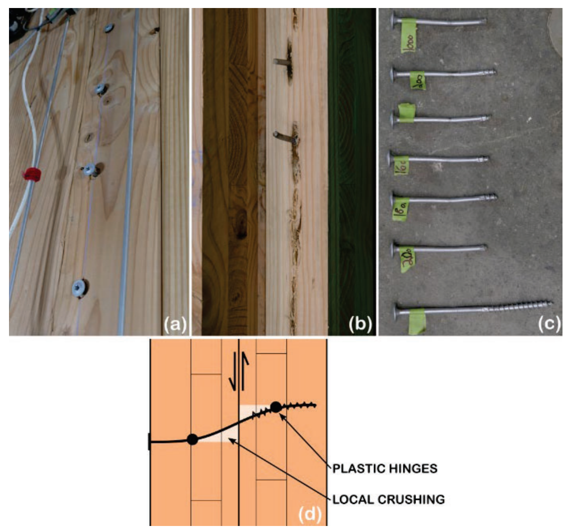

Figure 6: *Observed damage in screwed half-lap joint for LC walls: (a) Screw withdrawal, (b) local crushing inside half-lap. (c) removed screws, and (d) screw yielding mechanism.*

The BC wall experienced severe damage to the steel link beams, as shown in Figure 7. The webs yielded over the full length of the beams. In addition, the flanges adjacent to the end plate stiffeners yielded and eventually buckled inelastically. No damage was observed in the capacity-designed screwed end plate connections; therefore, the capacity design was successful.

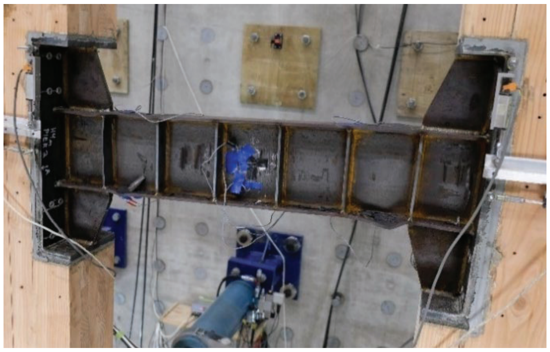

Figure 7: *Typical damage to steel link beams in beam-coupled wall specimen.*

4 COMPARISON OF RESULTS

Key experimental results, including wall strength, stiffness, ductility, and energy dissipation are summarized in Table 1. As expected, the single wall had the lowest strength, stiffness, and energy dissipation; however, it may provide the most simple and economic structural form for CLT shear walls. The LC and BC (coupled) walls demonstrated higher strength and stiffness and more energy dissipation. The displacement ductility factors (μ=D_u/D_y), varied between a minimum of 3.5 (LC1) and 7.6 (BC).

The LC walls had the lowest yield drift, which is beneficial because the system's hysteretic damping of the earthquake excitation (i.e., energy dissipation) begins at a lower drift level. As timber lateral systems are often drift-controlled, a low yield drift is advantageous for seismic design [15]. The average peak strength of LC1 normalized by its wall length (F_{max}/L) was 17% less than that of the single wall, despite having the same hold-downs and using twice as much timber volume. This was primarily due to the fastening pattern of its vertical joint in which screws were installed with 90° to the timber surface and with a 200mm spacing. The vertical joint was not sufficiently strong and stiff to promote a significant degree of coupling. LC2 was similar to LC1 except it used a 100mm screw spacing in the vertical joint. In contrast, it obtained 53% greater energy dissipation and peak strengths were 23% and 64% greater than those of LC1 in the positive and negative loading directions, respectively. Furthermore, its stiffnesses were 40% and 26% greater in the positive and negative directions, respectively. Apparently, more fasteners used in the vertical joint provided a more efficient lap-coupled wall type. However, caution must be exercised by the designer because if the half-lap joint is too strong, the energy dissipation of the system may be compromised.

The BC wall exhibited the highest strength, stiffness, drift capacity, and energy dissipation of the three wall types. This configuration involves a greater degree of fabrication and construction complexity. However, given its enhanced structural performance, it is likely that fewer walls would be required in a given building and therefore its application may be more economical than single or lap-coupled walls. Furthermore, it provides a practical wall configuration which can accommodate repetitive window or door openings in the architectural plans.

The force-displacement plots of the four wall specimens are shown in Figure 8. All wall specimens exhibited a ductile failure mode with damage concentrated in the designated regions described previously in Figure 1. The hysteresis loops show typical features of cyclically loaded timber connections: strength and stiffness degradation, and the pinching effect. The pinched behaviour is a result of cumulative local crushing around the yielding screws in the hold-downs and/or half-lap joints. The BC wall test showed the least amount of pinching due to the influence of the ductile link beams which yielded without exhibiting pinched behaviour on a component level.

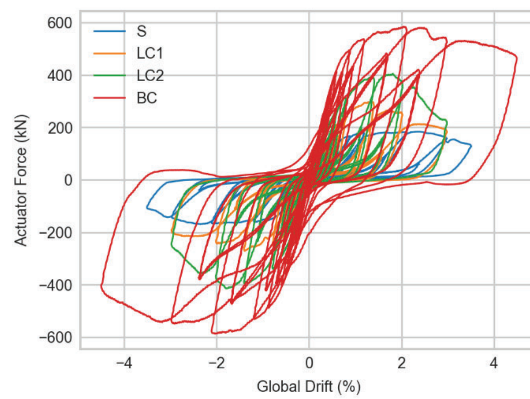

Figure 8: *Comparison of global drift vs. base shear.*

The energy dissipation of the primary and first two trailing cycles (characteristic of the CUREE loading protocol in Figure 4) are shown for each of the four wall tests in Figure 9. The pinched hysteretic behaviour and corresponding loss of energy dissipation is evident when comparing the amount of energy dissipation in the primary and trailing cycles for each wall type. Both the S and LC specimens exhibited minimal energy dissipation on their trailing cycles. However, the BC wall dissipated a reasonable amount of energy on its trailing cycles due to the influence of the steel link beams.

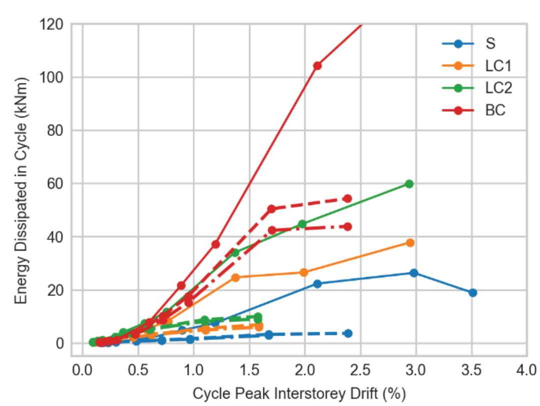

Figure 9: *Comparison of energy dissipation in each wall specimen.*

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Table 1: *Summary of wall test results*

	Single Wall (S)	Lap-Coupled Wall 1 (LC1)	Lap-Coupled Wall 2 (LC2)	Beam-Coupled Wall (BC)
D_{ref}^{a} (% Drift)	3.0%	2.0%	2.0%	3.0%
F_y (kN)	161 / -138	221 / -182	276 / -299	471 / -465
D_y (% Drift)	0.69% / -0.66%	0.40% / -0.32%	0.35% / -0.43%	0.58% / -0.56%
F_{max} (kN)	185 / -170	300 / -278	409 / -418	590 / -592
F_{max}/L (kN/m)	93 / -85	77 / -71	105 / -107	122 / -122
D_{Fmax} (% Drift)	2.31% / -2.38%	1.28% / -1.39%	1.70% / -1.78%	2.07% / -2.02%
F_u (kN)	140 / -132	236 / -200	317 / -315	470 / -465
D_u (% Drift)	2.99% / -3.17%	1.38% / -1.40%	1.98% / 2.02%	4.37% / -3.91%
K_i^{b} (kN/mm)	3.3 / 3.3	8.5 / 7.9	11.9 / 10.0	11.9 / 12.1
Displacement Ductility, μ	4.3 / 4.8	3.5 / 4.4	5.7 / 4.7	7.6 / 6.9
E^{c} (kNm)	80	154	236	482

[a]Reference displacement to define the CUREE loading protocol [14]
[b]Initial stiffness
[c]Cumulative energy dissipated up to 3% interstorey drift

5 CONCLUSIONS

This study conducted experimental testing on four balloon-type CLT shear walls with three wall configurations: a single wall, a coupled wall with screwed half lap joints, and a coupled wall with steel link beams. The cyclic performance of each specimen was evaluated and compared. Based on the test results, the following conclusions are drawn:

1. Mixed angle screwed hold-downs can provide a feasible, high-capacity hold-down connection for multi-storey CLT shear walls subjected to earthquake loading.
2. All three wall types in this study exhibited ductile failure modes and experimentally demonstrated their feasibility as lateral load resisting systems.
3. For the coupled walls with screwed half lap joints, the degree of wall coupling was governed by the screw spacing which can be optimized to achieve a balance of wall strength, stiffness, and energy dissipation.
4. Displacement ductility factors varied between the wall types, with the lowest value observed in the lap-coupled wall specimen LC1 and the greatest value observed in the beam-coupled wall specimen BC.
5. The beam-coupled wall BC with steel link beams demonstrated the greatest strength, stiffness, and energy dissipation, while also exhibiting the least amount of pinched hysteresis. However, it is more complex to fabricate and construct than the other wall types.

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