TIMBER CORE-WALLS FOR LATERAL LOAD RESISTANCE OF MULTI-STORY TIMBER BUILDINGS

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Cross-laminated timber, PresLam, seismic, stairwells, walls, XLam

ABSTRACT
This paper describes the results of experimental tests on post-tensioned Cross-Laminated Timber (CLT) core-walls tested under bi-directional quasi-static seismic loading. The half-scale two-storey test specimens included a stair with half-flight landings.

The use of CLT panels for multi-storey timber buildings is gaining popularity throughout the world, especially for residential construction. Post-tensioned timber core-walls for lift-shafts (elevator shafts) or stairwells can be used as tubular structures for resistance to seismic loads and wind loads in open-plan commercial office buildings.

Previous experimental testing has been done on the in-plane behaviour of single and coupled timber walls at the University of Canterbury and elsewhere. However, there has been very little research done on the 3D behaviour of timber walls that are orthogonal to each other, and no research to date into single post-tensioned CLT walls or CLT tubular structures.

This paper describes a “High Seismic option” consisting of full height post-tensioned CLT walls coupled with energy dissipating U-shaped Flexural Plates (UFPs) attached at the vertical joints between coupled wall panels and between wall panels and steel corner columns. An alternative “Low Seismic option” consists of post-tensioned CLT panels connected by screws, to provide a semi-rigid connection, allowing relative movement between the panels, producing some level of frictional energy dissipation. The Low Seismic option is suitable for wind loading in non-(or low-) seismic regions.

1 INTRODUCTION

Multi-storey timber structures are becoming increasingly desirable for architects and building owners due to their aesthetic and environmental benefits. In addition, there is increasing public pressure to have low damage structural systems with minimal business interruption after a moderate to severe seismic event.

Timber has been used extensively for low-rise residential structures in the past, but has been utilised much less for multi-storey structures, traditionally limited to residential type building layouts which use light timber framing and include many walls to form a lateral load resisting system. This is undesirable for multi-storey commercial buildings which need large open spaces providing building owners with versatility in their desired floor plan.

Options are needed for architects and engineers to utilise timber walls around stairwells and lift shafts (elevator shafts). To date, little research has been done on lateral load resistance of timber stairwell and lift shaft cores (especially for the post-tensioned PresLam system). There is an urgent need for stairwells to have more seismic resistance, following the potentially disastrous collapse of stairs in a number of buildings in the Christchurch earthquakes [1].

This paper describes the test results of two post-tensioned timber stairwells tested under bi-directional quasi-static seismic loading and provides recommendations for building designers.

2 MOTIVATION FOR RESEARCH

In research previously conducted [2, 3, 4] and
continuing at the University of Canterbury [5] the behaviour of post-tensioned timber (LVL) rocking and dissipative single and coupled walls, loaded in plane have been comprehensively investigated. The natural extension was to investigate how the Pres-Lam system could be incorporated into timber walls of different shapes, such as tubular structures for stairwells and lift shaft cores. This research intended to develop technical solutions for Pres-Lam core walls and used the experimental tests to validate the efficiency and practicality of the systems whilst providing practical guidelines for engineers and architects as to use such systems for a number of stairwell layout options.

The use of CLT is steadily growing in Europe [6] and Canada [7, 8] and is gaining traction in New Zealand since the commissioning of a CLT plant in Nelson by XLam Ltd. To date, the primary construction method for CLT panels is to use screws or similar mechanical fasteners as the connectors. Popovski and Karacabeyli [9] at FPInnovations in Canada investigated the seismic performance of CLT panels. A variety of panel and fastener layouts were investigated, however, all of the test specimens involved in-plane walls. No L or C shaped configurations were tested. The performance of each specimen was determined and typical results of the test specimens loaded cyclically are shown in Figure 1.

It is reported by various authors [6, 9] that experimental testing on CLT wall configurations showed that CLT structures can have adequate seismic performance when nails or screws are used with steel angle brackets. The definition of adequate seismic performance is in this case debatable. The hysteretic behaviour of the system is influenced heavily, and limited, by the behaviour of the mechanical fasteners at the base or edges of the wall panels. From Figure 1a, which shows typical hysteretic behaviour of nail type connections, it can be seen that there is a very significant amount of stiffness and strength degradation. For large displacement cycles, large amounts of slip appears to have occurred as the shear fasteners have pulled out from the CLT panels and provide little resistance to further displacement cycles. Typical damage to these types of fasteners is shown in Figure 1b and 1c, where the nails have withdrawn from the timber and yielded. Therefore, following a large earthquake, the structural system would be left with greatly reduced strength and stiffness capacities.

This research was aimed at developing seismic resisting stairwells, which could be either fully independent or detached from the main structure, or act as core-walls for the whole or portion of a=the main structure.

3 EXPERIMENTAL TESTING

Two, two-storey, ½ scale, stairwell cores were tested under uni and bi-directional quasi-static cyclic loading. The first specimen was referred to as the Low Seismic option consisting of post-tensioned CLT walls screwed together. The second specimen, referred to as the High Seismic option, comprised of post-tensioned CLT walls coupled with UFP devices. Various tests were performed on each specimen, altering the post-tensioned force and connection details. The test setup for both options is shown in Figure 2; 2a shows loading in the X direction and 2b shows loading in the Y direction.

3.1 Low Seismic Option

The primary objective of the Low Seismic test was to investigate the performance of the rocking system with simple screwed connections. These connections allowed rapid construction without high demands
for precise tolerances. The layout of this specimen consisted of a rectangular tube with two sets of coupled walls in the longitudinal direction and two sets of single walls (one with doorway openings), in the transverse direction as shown in Figure 3a.

Horizontal screws connected the coupled and perpendicular panels (Figure 3b). The screws provided a semi-rigid connection, allowing a relative movement between the wall panels. The relative movement caused deformation in the screws which acted as ductile fuses. Seven-wire tendons were used to post-tension the CLT panels as shown in Figure 3c.

Beams representing the floor slab (Figure 4) were connected to the walls, such that there were two beams running in the long direction and short direction of the stairwell. The beams represent gravity and drag beams respectively and are part of the floor diaphragm transferring horizontal loads into the staircase. The beams running along the short direction were continuous; therefore the beams in the long direction had to be spliced with steel plates and rivets (see Figure 14). The drag beams were connected directly to the walls with rings of bolts forming an approximate pinned connection near the centre of each wall (Figure 5). As the rocking motion of the wall imposes uplift and rotation to the beams as shown in Figure 4, special considerations for the design of the connections are necessary. This displacement and rotation incompatibility was minimized by the geometry of the connection (closely spaced bolts) and the flexibility of the connectors. Vertical uplift of the walls was imposed into the beams creating differential movement at the connection between the drag beams running perpendicular to each other. The out-of-plane flexibility of the steel plate allowed for this movement while working in the elastic range and was able to transfer the horizontal forces throughout the testing.
was also investigated as part of a larger study into floor and diaphragm behaviour [11].

Similar loading beams to those used for the Low Seismic specimen were used for the High Seismic specimen; however, the loading beams were connected to SHS corner columns rather than directly into each wall, to minimise vertical deformations of the flooring system as the walls rock. A sketch of the behaviour of the loading beams is shown in Figure 7. As the walls rock, the loading beams, and hence floor slab, remain level because the SHS corner columns do not undergo any vertical movement. The force transfer was guaranteed by a single bolt connected directly to the steel column and via a reinforcing steel plate to the beams. The steel plate was connected to the drag beam by rivets and allowed for the force transfer without relying on the low embedment strength of timber. Figure 8a shows the pinned bolted connection between the SHS column and a loading beam.

![Figure 7: Behaviour of the loading beams for the High Seismic option, as uplift occurs in the walls](image)

3.2 High Seismic Option

The lateral load resisting system of the High Seismic specimen comprised of post-tensioned rocking CLT walls coupled with energy dissipation devices. Energy dissipaters were used in the form of U-shaped Flexural Plates (UFPs) attached between wall panels and the square hollow section corner columns as shown in Figure 6. Construction was slower than for the Low Seismic option because of the very tight tolerances required for all connections between prefabricated components.

The focus of the testing was on the construction of the system and its overall seismic behaviour. Similar research investigating the performance of rocking walls with end columns has been carried out [10]. Reduction of damage to the floors and loading beams

![Figure 6: L High Seismic option: a) Post-tensioned CLT walls with steel corner columns; b) UFP connection details between coupled walls; c) UFP-Steel corner column connection](image)

was also investigated as part of a larger study into floor and diaphragm behaviour [11].

In the High Seismic tests, there was no direct connection between the loading beams and the wall panels, so that all lateral loads from the beams had to be transferred into the walls by direct bearing of the steel column to the edge of the timber wall panel. This high local load induced considerable friction when the steel column was sliding vertically against the edge of the rocking wall panel. If this friction needs to be minimized, special bearings with low friction materials can be used to transfer the loads into the wall.

4 RESULTS AND DISCUSSION

4.1 Low Seismic Option

For the purposes of this experimental investigation, six tests were undertaken. Each test had three primary variables: the post-tensioned force, the number of screws per joint, and the type of loading. The tests
Figure 8: a) Pin connection between SHS and loading beams, during construction. b) Base of steel column

performed are outlined in Table 1. Two seven-wire tendons were used in each wall.

Table 1: Summary of Low Seismic test specimens

<table>
<thead>
<tr>
<th>Test</th>
<th>Post-Tensioning</th>
<th>Screws / Joint</th>
<th>Loading</th>
<th>Max Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Low (40kN/tendon)</td>
<td>3</td>
<td>X/Y Separate</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>Low (40kN/tendon)</td>
<td>20</td>
<td>X/Y Separate</td>
<td>0.5</td>
</tr>
<tr>
<td>3</td>
<td>High (100kN/tendon)</td>
<td>3</td>
<td>X/Y Separate</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>High (100kN/tendon)</td>
<td>3</td>
<td>Clover</td>
<td>1.25</td>
</tr>
<tr>
<td>5</td>
<td>High (100kN/tendon)</td>
<td>20</td>
<td>X/Y Separate</td>
<td>1.25</td>
</tr>
<tr>
<td>6</td>
<td>High (100kN/tendon)</td>
<td>20</td>
<td>Clover</td>
<td>1.05</td>
</tr>
</tbody>
</table>

Selected test results displaying overall force-displacement hysteretic response of the Low Seismic specimen are shown in Figure 9, with full results discussed in [12]. Figure 9a and 9c show the results of coupled and single walls respectively for Test 1 with a low post-tensioning force and a low number of screws (3) per joint. Figure 9b and 9d show the results of coupled and single walls respectively for Test 5 with a high post-tensioning force and a high number of screws (20) per joint. The results of the test on the coupled walls have been filtered to remove horizontal slip which occurred in the test rig. Therefore, the maximum drift that is shown in the Figure 9, in some cases, is less than the maximum input drift from Table 1.

For each test, the coupled walls behaved noticeably different from that of the single walls. The single post-tensioned walls for test configurations with a low number of screws (Figure 9c) displayed the hysteretic behaviour of a wall with only unbonded post-tensioning. This behaviour is characterised by a bi-linear elastic response with no energy dissipation. Full re-centring behaviour was observed for these test configurations. For test configurations with a large number of screws (Figure 9d), slightly more energy dissipation was observed in the hysteretic behaviour.

For all of the tests, significantly more energy dissipation was observed in the hysteretic behaviour of the pairs of coupled walls than the single walls. Even for the tests with a low number of screws, a large amount of energy dissipation was observed. Whilst some of the energy dissipation was provided by the deformation of screws (Figure 10), the majority was produced by friction in the vertical joint between the two adjacent wall panels. For the high screws tests (Figure 9b), the effect of having a large number of screws was to restrict the relative movement between the coupled panels. With a low number of screws, the panels were able to ‘rock’ freely, relative to one another. The large number of screws locked the panels together. A greater strength and stiffness was observed for the high screw configurations. However this produced large horizontal forces at the toe of the wall, resulting in inelastic crushing of the timber as shown in Figure 11.
Figure 9: Selected results from Low Seismic tests:
a) Coupled walls - Low PT, Low screws, X direction;
b) Coupled walls - High PT, High screws, X direction;
c) Single wall - Low PT, Low screws, Y direction;
d) Single wall - High PT, High screws, Y direction.

Figure 10: Deformation of screws at perpendicular panels following Test 1 (low screw configuration)

Figure 11: Crushing at the toe of a wall and deformation of the shear key during Test 5 (high screw configuration)

During testing the rotation and uplift of the drag beams were measured. Because of the pin-like connection to the wall, only little rotation was imposed into the beams. The uplift of the wall however, resulted in beam bending and differential vertical movement of the drag beams running perpendicular to each other. Visual inspections showed that none of the connections (beam-to-wall and beam splice) underwent plastic deformation. Figure 14 shows the out-of-plane bending of the connection steel plate during testing. The plate was able to transfer the load throughout testing and did not have any residual deformation. For a real structure with a timber floor diaphragm, wooden panels would be nailed or screwed to the beams. These would need to follow the bending of the beams and the differential movement at the intersection of the drag beams. Generally the bending of the beams can be accommodated by the panels, this might not be the case for the differential movement as nails or screws can be pulled out. It is therefore suggested not to rely on the shear transfer of the panels in the direct vicinity of the intersection of the drag beams but to transfer all necessary diaphragm shear along the undisturbed parts of the beams.

For floor layouts with higher out-of-plane stiffness or when localized damage shall be avoided, other connections between the wall and the drag beams should be adopted. Moroder et. al. 2014 [11] tested successfully, connections with inclined fully threaded screws and steel-to-steel connections with pins in slotted holes which minimized rotation and vertical displacement incompatibilities while still transferring the horizontal forces.

4.1 High Seismic Option

For the purposes of this investigation, seven tests were undertaken. In a similar fashion to the Low Seismic tests, each test had three primary variables: the post-tensioned force, the number of dissipaters and the type of loading. A summary of the High Seismic test schedule is shown in Table 2. Two seven-wire tendons were used to post-tension each wall. Each UFP was designed to provide 30kN of shear force.

Selected test results of the High Seismic specimen are shown in Figure 12, with full results discussed in [12]. Figure 12a and 12b show the results of coupled and single walls respectively for Test 4 and Test 5. Figures 13a and 13b show the results of coupled and single walls for Tests 6 and 7 respectively. In all cases there is a large amount of friction because all the lateral loads are transferred into the wall panels by direct bearing on a wood-to-steel interface where vertical sliding occurs during rocking of the walls. The results of the test on the coupled walls have been filtered to remove horizontal slip which occurred in the test rig. Therefore, the maximum drift that is shown in the Figure 12, in some cases, is less than the maximum input drift from Table 1.
Table 2: Summary of High Seismic test specimens

<table>
<thead>
<tr>
<th>Test</th>
<th>Post-tensioning</th>
<th>UFPs / joint</th>
<th>Loading</th>
<th>Max Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Low (40kN/tendon)</td>
<td>2</td>
<td>X/Y Separate</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>Low (40kN/tendon)</td>
<td>2</td>
<td>Clover</td>
<td>1.05</td>
</tr>
<tr>
<td>3</td>
<td>High (100kN/tendon)</td>
<td>2</td>
<td>X/Y Separate</td>
<td>1.0</td>
</tr>
<tr>
<td>4</td>
<td>High (100kN/tendon)</td>
<td>1</td>
<td>X only</td>
<td>1.25</td>
</tr>
<tr>
<td>5</td>
<td>High (100kN/tendon)</td>
<td>0</td>
<td>Y only</td>
<td>1.75</td>
</tr>
<tr>
<td>6</td>
<td>High (100kN/tendon)</td>
<td>2</td>
<td>Y only</td>
<td>3.5</td>
</tr>
<tr>
<td>7</td>
<td>High (100kN/tendon)</td>
<td>0</td>
<td>X only</td>
<td>3</td>
</tr>
</tbody>
</table>

Figure 12: Results from testing of the High Seismic specimen for a) Test 4 (X-only), b) Test 5 (Y-only)

For each test, the hysteretic behaviour of the coupled walls was significantly different to that of the single walls (Figure 12). The response of the coupled walls was influenced significantly by additional friction between adjacent panels. Comparatively, the UFP devices did not have a significant effect on the energy dissipation of the coupled walls, as the behaviour was dominated by the friction contribution. However, the UFP devices did contribute to the strength and stiffness, particularly in the single wall case.

Further tests were performed where the UFP devices were removed from the coupled walls (Figure 13a) and the single walls (Figure 13b). A reduction in stiffness and strength was observed, predominantly in the response of the single walls. For all walls, especially the coupled walls, a significant amount of energy dissipation was achieved. The single walls displayed the more conventional hybrid hysteretic behaviour, with a bi-linear backbone, energy dissipation and re-centring properties. For the single walls, no stiffness or strength degradation occurred. However, some stiffness degradation was observed in the response of the coupled walls.

Figure 13: Results from testing of the High Seismic specimen for a) Test 6 and, b) Test 7 (note different scales)

Also for the High Seismic option beam rotation and uplift were measured. Since the beams were connected to the corner columns by a single pin, only very little rotation and uplift was imposed into the beams. The whole floor diaphragm would therefore remain level.

5 CONCLUSIONS AND DESIGN RECOMMENDATIONS

5.1 Conclusions

A series of experimental quasi-static cyclic testing under uni- and bi-directional cyclic loading were performed on two, ½ scale, two-storey stairwell cores. Key points from the results of the Low Seismic and High Seismic specimens are shown below.
5.1.1 Low Seismic Option

- The construction of the Low Seismic specimens was simple and rapid, enabled by the prefabrication of the CLT panels and simple screw connections between them. Accurate tolerances were not a concern.

- The Low Seismic specimens were tested with both a high and low numbers of screws.
  - The low screw configurations produced the best seismic behaviour, with considerable sliding displacement at the junctions between panels.
  - The high screw configurations gave increased stiffness and strength (150kN to 250kN), but restricted the amount of uplift, causing lateral sliding displacement and some crushing at the base of the walls. These configurations would be better suited to an elastic (rather than ductile) seismic design procedure.

- Regardless of the number of screws, recentering, due to the post-tensioning, occurred after all levels of lateral loading.

- For both low and high screw configurations, very little energy dissipation was provided by the deformation of the screws. For the Low Seismic tests, a large amount of energy dissipation was generated from friction at the vertical joint between the two coupled wall panels in the same plane, and less from the joints between panels at the corners of the stairwell.

- Closely spaced bolts worked almost like a hinged connection between the walls and the beams, thus reducing the imposed rotations to the drag beams. Uplift of the walls however caused some bending of the beams and differential vertical movement at their intersection. The beam splice therefore needs to allow for this movement without compromising the load transfer. A simple steel plate connected at a sufficient distance from the beam end with rivets proved to work successfully. Diaphragm panel elements situated around this disturbed area might have their connections compromised; the shear transfer to the beams should therefore be relied on connections further away from the disturbed area.

5.1.2 High Seismic Option

- The speed of construction of the High Seismic specimen was limited by the high degree of accuracy required in all connections between pre-fabricated elements, especially between the corner columns and the foundation, the panels and columns, the coupled wall connections, and the UFPs.

- Good hysteretic response was observed with excellent energy dissipation and re-centring in all tests. The energy dissipation contribution to the total hysteretic behaviour was significantly influenced by friction between adjacent elements. The friction component of the energy dissipation was greater for the coupled walls than for single walls. Even for tests where all the UFPs were removed, there was a significant amount of energy dissipation from friction alone.

- The steel corner columns were very effective in isolating the floor system from the uplift and rotation of the rocking walls. Minimal vertical displacement of the loading beams was observed during testing. Careful design of the beam-to-column connection is necessary as big forces need to be transferred while allowing for the rotation. The corner columns were also very effective in acting as shear keys for the panels.

5.1.3 Wind Loading

- The best option for extreme wind loading is the “Low Seismic” option with a high configuration of screws to increase the lateral stiffness and strength of the CLT core.

5.2 Recommendations for Building Designers

- For buildings in high seismic regions, the “High Seismic” option using post-tensioned walls and steel columns coupled with UFPs will give the best seismic performance. An alternative solution could use a “Low Seismic” option modified with additional dissipation devices.

- For buildings located in low seismic areas, the “Low Seismic” option will result in a very cost-
effective design, using post-tensioned walls connected together with screws.

- For locations governed by wind loading, the “Low Seismic” option with a high concentration of screws will provide excellent performance.

- All lateral load resisting walls must have appropriate shear keys to prevent horizontal sliding of the panels on the foundations, and careful detailing of floor to wall connections to ensure efficient transfer of vertical and horizontal loads, without any damage during extreme events.

- Uplift and rotation incompatibilities can be minimized by connecting the drag beams with closely spaced bolts to the wall. Splices for drag beams around the core walls should allow for differential vertical movements. Floor panels should transfer the shear forces to the drag beam away from the splice. For stiffer out-of-plane floors, connections between the walls and the beams steel plates with a pin in slotted holes or inclined fully threaded screws are suggested. To minimize the incompatibilities even further, the beams should be connected to corner columns by a single pin.

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REFERENCES


