Global Response of a Two Storey Pres-Lam Timber Building

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ABSTRACT: Recent structural timber innovations at the University of Canterbury have let to the construction and experimental testing of a large scale, 2 storey, post-tensioned timber frame and wall building. The building was subjected to uni-directional and bi-directional quasi-static seismic testing, up to a maximum drift of 3.0%. The influence of concrete diaphragms and additional mild steel reinforcement for frames and walls are examined. For these tests, the structure responded essentially elastically. The addition of a thin concrete diaphragm had limited effect on the hysteretic response of the frames and walls, providing a 15% and 25% increase in strength respectively at design displacements (2% drift). Compressive deformation of beam-column connections resulted in minimal beam elongation, avoiding damage to the concrete slab. Additional mild steel reinforcement across the beam-column connections had little effect on the lateral resistance of the frames at 2% drift but was more effective at 3% drift. Simultaneous bi-directional loading induced a minor increase in the in-plane resistance of the frame and wall systems.

1 INTRODUCTION

New structural systems for multi-storey timber buildings are under development at the University of Canterbury in collaboration with the Structural Timber Innovation Company (STIC Ltd). These systems, referred to as Pres-Lam, are suitable for a wide range of building types, including commercial structures. They have the potential to compete with existing forms of construction in terms of cost, flexibility of structural form and structural performance (Buchanan et al. 2008). The Pres-Lam system incorporates large timber structural frames or walls, constructed of Laminated Veneer Lumber (LVL), connected by steel post-tensioning (see Figure 1). This timber connection technique (Palermo et al. 2005) was adapted from post-tensioned pre-cast concrete systems (Pampanin 2005; Priestley et al. 1999). For seismic applications, the combination of timber and post-tensioning is particularly efficient since it avoids potential brittle failure modes that occur in traditional timber solutions (Buchanan and Fairweather 1993). Pres-Lam systems fit well into Performance-Based Seismic Engineering (PBSE) design (Christopoulos and Pampanin 2004) because residual deformations and structural damage are minimized.

The seismic response of Pres-Lam frames is strongly dependent on the detailing of the beam-column connections (Newcombe et al. 2008a; Newcombe et al. 2008b). Potential issues arise from perpendicular-to-grain loading in the columns, which could result in creep, low connection stiffness and shear deformation of the joint panel region. The energy dissipation capability of the structural system can be improved with the addition of external mild steel devices to the beam-column connections, known as the ‘Hybrid’ connection (NZS3101:2006; Stanton et al. 1997), because it combines the non-linear elastic and re-centering capabilities of post-tensioning with the energy dissipating properties of mild steel. However, previous research (Newcombe et al. 2008b), has indicated that, depending on the connection detailing, the addition of mild steel reinforcement could give only minor increases to hysteretic energy dissipation (Priestley et al. 2007) for the timber frame system because of the larger elastic
deformations of the timber elements compared to concrete. This paper describes experimental tests on the two-storey post-tensioned timber building shown in Figure 2.

![Figure 1. The Pres-Lam system concept implemented into a beam-column frame connection](image)

2 EXPERIMENTAL ARRANGEMENT

The building consists of independent frames and walls in each direction and timber-concrete composite floors. Initially, it was constructed excluding a floor diaphragm. Quasi-static cyclic loading (according to ACI T1.1-01, 2001) was applied to the columns (Stage 1). Subsequently, a concrete slab floor diaphragm was poured and loading was applied to the frames and walls via the floor (Stage 2), both uni-directional and simultaneous bi-directional cyclic loading.

2.1 The Pres-Lam Building

The test building (see Figure 2) is a 2/3 scale model. The beams and columns are connected internally by 4 - 12.7mm (0.5 inch) post-tensioning tendons (Figure 2c). External mild steel reinforcement (Pampain 2005) connects the column bases to steel foundations. For some tests, external mild steel reinforcement is added to the beam-column connection, as depicted in Figure 2f, to increase the frame stiffness at small displacements (serviceability limit state), to provide mechanical damping and increased strength at large displacements (ultimate limit state). Internal steel plates reinforce the beam-column joints at level 2 and large SPAX screws are used to reinforce the joint region at level 3. All the beams and columns have a cross-section of 254 x 400mm.

The walls are connected to steel foundations by 5 – 12.7mm tendons and have a constant cross-section of 144 x 800mm (Figure 2d). Post-tensioned only and hybrid wall systems are tested with U-shaped flexural plates (UFPs) used to couple adjacent walls (Kelly et al. 1972). For stage 2, the frames and the walls are connected to the concrete slab as shown in Figure 2e. The frames and walls are designed to remain elastic up to the design displacement of 2%. Hence, the yielding strain of the timber is not exceeded within the rocking connections.

2.2 The Test Apparatus

Quasi-static cyclic loading was applied to the structure via four hydraulic rams, two in each direction (see Figure 2b).

Forces were applied to the structure in the ratio of 2 to 1 for Levels 3 and 2 respectively. For Stage 1 (before the concrete diaphragm is cast), load was applied to one of the exterior columns (see Figure 2g and 2i). For Stage 2 (with the concrete floor), load is applied to the diaphragm by steel plates bolted to the slab (see Figure 2b, 2h and 2j). This loading simulates seismic inertial forces and tests the diaphragm-to-frame connections (Figure 2e) while allowing beam elongation.
Figure 2. Details of test building; a) Bare Building b) Building and test apparatus c) Frame elevation d) Wall elevation e) Concrete slab details (Newcombe et al. 2009) f) Additional mild steel reinforcement g) Test apparatus, Stage 1 elevation view h) Test apparatus, Stage 2 elevation view i) Test apparatus, Stage 1 plan view j) Test apparatus, Stage 2 plan view

(Rams not shown)
2.3 The Loading Protocol

Initially, the building was subjected to uni-directional cyclic loading in both the frame (x) and wall (y) directions (see Figure 2j) up to a peak drift of 2.0%.

For the final test, simultaneous loading in the x and y directions was applied to the building up to a peak drift of 3.0%. At each drift amplitude, one bi-directional (four-leaf-clover) displacement pattern was completed, followed by uni-directional cycles in x and y.

3 EXPERIMENTAL RESULTS

3.1 Uni-directional Frame Response

Figures 3a and 3b show the global hysteretic response, plotting the total combined base moment for both frames versus the building drift. These figures demonstrate that the frames remain essentially elastic with little hysteretic energy dissipation up to the targeted level of displacement. There is no significant loss in strength or stiffness for repeated cycles.

Figure 3a shows that the addition of a concrete slab has little effect on the strength of the building in the frame direction. There are three feasible mechanisms in which the concrete slab may cause an increase in the strength; localized bending of the slab, slab induced axial forces which increase the strength of the beam-column connections and resistance provide by out-of-plane walls. Experimental instrumentation shows that the bending moment in the beam-column connections did not increase due to slab interaction. Taking the total base moment and subtracting the out-of-plane moment resistance of the walls, the base moment contribution caused by the interaction of the slab with the building is approximately 15% (at 2% drift). Therefore, it is likely that the additional strength of the building in the frame direction is caused by local bending of the floor slab around the exterior columns.

Figure 3b shows the hysteretic response of the post-tensioning only (P.T. Only) and hybrid connections. At 2% drift, the adopted mild steel reinforcement had little effect on the resistance of the frame providing additional strength of approximately 10%. The elastic deformation of the frames delayed the gap opening, and consequent activation of the mild steel reinforcement, until over 1% drift. The mild steel began to yield at approximately 1.5% drift, so that the yielding (and hysteretic damping) remained low. Some deformation in the anchorage of the external mild steel devices meant that there was no increase in the initial stiffness for the Hybrid system.

3.2 Uni-directional Wall Response

The wall elements remain elastic up to 2% drift (see Figure 3c and 3d). The total capacity of the wall system was 35% higher than the combined design base moment capacity of the walls and columns. This is attributed to the coupling action of the concrete slab and edge beams.

From Figure 3d it is evident that the coupling effect from the edge beams provided only an additional 8% base moment. This indicates that coupling created by localized bending of the concrete slab resulted in a significant increase in lateral resistance of the building in the wall direction (approximately 25%). The short spans between the walls and columns caused high curvature in the slab, amplifying this coupling action.

As shown in Figure 3d, negligible hysteretic damping is generated in the ‘post-tensioning only’ tests. For the hybrid test, the UFPs were designed to provide roughly half the base moment of the post-tensioned walls, resulting in high hysteretic damping of approximately 22% equivalent viscous damping (EVD) (Priestley et al, (2007)). However the test results showed that the UFPs provided only 28% of the base moment. Hence, the coupling action increased the strength of the system but reduced the hysteretic damping of the wall system by 35% (to approximately 14% EVD).
3.3 Bi-directional Response

In Figure 4a and 4b, the base moment in the frame (x) and wall (y) directions are compared for uni-directional and bi-directional loading at 2.0% drift. It is apparent that there is only a minor increase in lateral resistance for bi-directional loading, with the largest difference in the wall direction.

During the bi-directional testing the building was subject to a peak drift of 3.0% (see Figure 4c and 4d). In contrast with Figure 3b and 3d, the energy dissipation devices for the frames and walls are more effective at larger displacements, providing a significant increase in strength (35% and 50% respectively) and more hysteretic energy dissipation (9% and 12% EVD damping respectively).
3.4 **Damage observations**

3.4.1 *Design Level Displacements (2% drift)*

Up to drifts of 2%, there was no significant damage to the building. The timber surrounding all beam-column connections remained elastic (see Figure 5a and b). For stage 1 testing, minor damage was induced at the beam-column connections adjacent to column C1 (see Figure 2g), caused by the additional axial loads from the testing apparatus.

For uni-directional testing in the frame direction, minor cracking occurred in the concrete slab (see Figure 5c), none of which was due to geometrical frame elongation. Localized cracking was observed adjacent to the walls due to displacement incompatibility because the floor remained flat as the walls rotated out-of-plane. Other cracks were induced in the slab when subjected to tension by the testing apparatus for negative displacements. The total elongation of the slab at 2% drift was 1.3mm with a residual elongation of 0.4mm, all concentrated around the walls. More cracking occurred under uni-directional testing in the wall direction, with maximum crack widths of 0.2mm at 2.0% drift. The level of cracking in the wall direction was magnified by the short span between columns and walls.

![Graphs showing hysteretic response](image)

*Figure 4. Global hysteretic response; a) Frame system uni- and bi-directional response b) Wall system uni- and bi-directional response c) Frame system with and without additional mild steel at 3% drift d) Wall system with and without UFP energy dissipation at 3% drift*
3.4.2 Beyond Design Level Displacements (3% drift)

The bi-directional testing up to 3% drift induced more cracking in the concrete slab (maximum crack width 0.5mm). The total residual elongation of the slab in the frame direction was 1.6mm, again localized around the walls.

At 3% drift, an internal column fractured at Level 3 (see Figure 5d and 5e) when the moment demand on the column was approximately 100 kN.m, less than half the design flexural capacity (240 kN.m). It is likely that the failure was induced by high stress concentrations around the anchorage pins for the external steel reinforcement (see Figure 2f) combined with column bending stresses. It was not possible to meet the code-minimum edge distances for the anchorage pins (NZS3603:1999), so screws were used to reinforce the timber adjacent to the pins. The screw reinforcement may have contributed to the fracture.

There was no apparent damage to the wall systems in any tests. However, the gap at the base of walls increased. This allowed the radius of the UFPs to enlarge, reducing their effectiveness at small drifts, causing a reduction in inelastic strains and energy dissipation.

Figure 5. Damage observations; a) Level 2 beam-column joints at 2% drift b) Level 3 beam-column joints at 2% drift c) Crack pattern of Level 2 floor slab d) Column fracture at 3% drift, front e) Column fracture at 3% drift, back f) UFP energy dissipater at 3% drift

4 CONCLUSIONS

- Experimental tests on a two-storey post-tensioned timber frame building gave excellent seismic performance, with full re-centering and no significant damage at 2% drift.
- The high proportion of elastic deformation in the frame at 2% drift resulted in only limited yielding of mild steel reinforcement at the beam-column connections. A higher level of yielding and energy dissipation is achieved at larger displacements (3% drift).
- Compressive deformation of the timber beam-connections limited the overall frame elongation and therefore the damage to the concrete slab. In turn, this limited the interaction of floor slab and the building in the frame direction, providing an increase in base moment of only 15%.
- The interaction of the concrete slab and the building in the wall direction was more significant than the frames direction (increasing the base moment by 25%). In real structures it is likely that the interaction of the slab will be less significant since walls and columns will be spaced at larger distances.
Simultaneous bi-directional loading had little effect on the in plane resistance of the frames and walls, slightly increasing their capacity.

Stress concentrations around energy dissipation anchorages caused one column to fracture at 3% drift. Further research is necessary on the effects of anchorages in highly stress beam-column joints.

REFERENCES

American Concrete Institute Innovation Task Group 1 and Collaborators (2001). Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary, ACI T1.1-01 and ACI T1.1R-01, Farmington Hills, MI: American Concrete Institute


