Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1447

EXPERIMENTAL TESTING OF A TWO-STOREY POST-TENSIONED TIMBER BUILDING

M. P. Newcombe[1], S. Pampanin[2] and A. H. Buchanan[3]

ABSTRACT

Recent structural timber innovations in New Zealand have let to the construction and experimental testing of a large scale, 2 storey, post-tensioned timber building. The building is subjected to quasi-static cyclic seismic testing up to design level Drifts of 2%. The influence of concrete diaphragms, additional mild steel reinforcement and column reinforcement are examined. For these tests, the structure responds essentially elastically. The addition of a thin concrete diaphragm has little effect on the hysteretic response of the frames, providing only a 15% increase in strength. At the design level displacements for this specific test-configuration, additional mild steel reinforcement across the beam-column connections has little effect on the lateral resistance of the frames. However, the response is significantly influenced by joint panel reinforcement and column face armoring. The compressive deformation of beam-column connections resulted in minimal beam elongation, resulting in very little damage to the concrete slab.

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 Pre-Lam, are suitable for a wide range of building types, including commercial structures, which 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 tendons (see Figure 1). Originally, these connection techniques (Palermo *et al.* 2005) were adapted from post-tensioned pre-cast concrete systems (Pampanin 2005; Priestley *et al.* 1999) which avoided significant inelastic deformations in the (plastic hinge zones of) structural elements by localizing deformation within the joint regions between beams and columns. For seismic applications, the combination of timber and post-tensioning is particularly efficient since it avoids potential brittle failure modes in traditional timber solutions (Buchanan and

[1]PhD Candidate, [2] Assoc. Professor and [3] Professor, Dept. of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand.

Fairweather 1993). Pre-Lam systems fits well into current Performance-Based Seismic Engineering (PBSE) philosophies (Christopoulos and Pampanin 2004) since residual deformations and structural damage are minimized.

This paper describes a series of preliminary experimental tests performed on a two-storey post-tensioned timber building shown in
Figure 2a. The building consists of independent frames and walls in each direction and timber-concrete composite floor units. This paper reports the first stage of testing, which investigates the building response in the frame direction, also the influence of a concrete diaphragm and various beam-column connection details.

For traditional structural concrete frames, the concrete floor slab can significant influence the lateral load resistance (Peng *et al.* 2006) resulting in significant damage under cyclic earthquake loading. This study assesses the effect of concrete slabs on post-tensioned timber frame behavior, diaphragm connections (Newcombe *et al.* 2009) and floor supports.

The seismic response of Pres-Lam frames is strongly dependent on the detailing of the beam-column connections (Newcombe *et al.* 2008a). Potential issues arise from perpendicular-to-grain loading in the columns, where the columns connect with the beams, which could result in creep, low connection stiffness and shear deformation of the joint panel region. Of particular concern is the joint panel shear deformation, predicted to contribute to approximately 40% of the total frame deformation (Newcombe *et al.* 2008b). Beam-column connection details have been specifically designed to reduce and minimize these effects.

The energy dissipation capability of the structural system can be improved with the addition of external mild steel devices to the post-tensioned beam-column connection. This solution is widely 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.

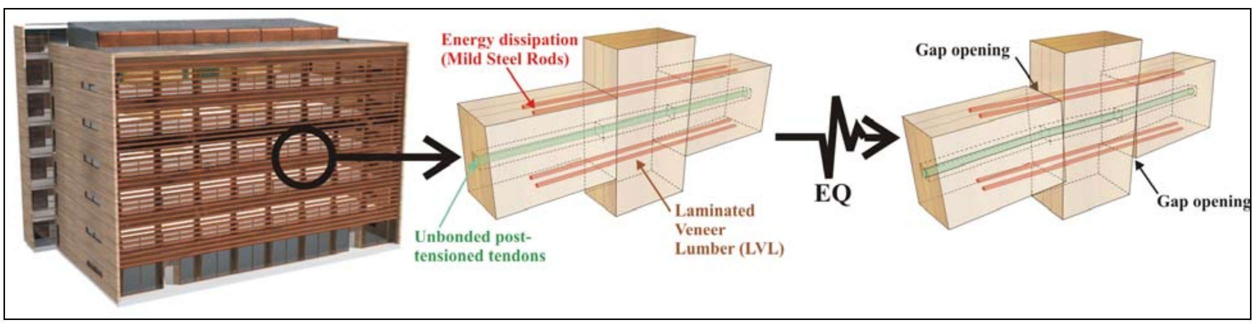

Figure 1. The Pres-Lam system concept implemented into a beam-column frame connection

Experimental Arrangement

Two distinct experimental arrangements were used during the frame testing. Initially, the building was constructed excluding a floor diaphragm. Quasi-static cyclic loading (according to ACI T1.1-01, 2001) was applied the columns (Phase 1). Subsequently, a concrete slab floor diaphragm was poured and loading was applied to the frames via the floor (Phase 2).

The Pres-Lam Test-Building

The building is a 2/3 scale model, with independent frames and walls in each direction (see Figure 2a) and timber-concrete composite floor units. This paper will focus on the response of the structural frames, shown in Figure 2b. The beam and columns are connected by four 12.7mm (0.5 inch) post-tensioning tendons. At the base of the structure, external mild steel reinforcement (referred to as "plug&play" energy dissipation for their easy replacebility, see Pampanin, 2005) connects to the columns to steel foundations which are in turn connected to the laboratory strong floor. All the beams and columns have a constant cross-section of 254 x 400mm.

Phase 1 testing has no concrete diaphragm. In Phase 2, a concrete slab is poured and connected to the frames, as illustrated in Figure 2c and the seismic loads are applied to the concrete topping, diverted into an edge joist via notched composite connections and coach screws, and transferred to the seismic beams via wood screws. This diaphragm connection is designed to accommodate seismic deformations (such as beam elongation) in the deformable timber joist-to-beam connections.

Additional external mild steel reinforcement is added to the beam-column connection (creating a 'Hybrid' connection) during both phases of the testing, as depicted in Figure 2d. The central region is necked down to 10mm in diameter to localize the inelastic deformation within the 'fuse length', away from the connections. To avoid buckling in compression an epoxy filled anti-buckling tube encases the fuse length. These devices are designed to provide additional stiffness at small displacements (serviceability limit state), mechanical damping and strength at large displacements (ultimate limit state).

Two beam-column connection details are used. On Level 2, steel plates reinforce the column and beam-column connection (see Figure 2e). The steel plate arrangement consists of an internal box section, design to prevent long term creep deformation and initial elastic shortening and to increase the joint panel shear stiffness. The end plates armour the column face against perpendicular-to-grain compression and increase the connection stiffness. On Level 3, large wood screws (SPAX screws) are used to reinforce the joint region (Figure 2f). The screws are designed to resist long term creep and initial elastic shortening deformation plus increase the joint panel shear stiffness.

The frames are designed to remain elastic up to the design level rotation of 2%. Hence, the yielding strain of the timber is not exceeded within the beam-column connections.

The Testing Apparatus

Quasi-static cyclic loading is applied to the structure via two 100 tonne hydraulic rams. The force is applied to the structure in the ratio of 2 to 1 for Level 3 and 2 respectively. This inherently assumes a linear displacement profile and uniform mass distribution. For phase 1 (before the concrete diaphragm is cast) load is applied to the one of the exterior columns (column C1 in Figure 3a and 3b). For phase 2, with the concrete floor present, load is applied to the diaphragm by steel plates that are bolted to the slab (see Figure 3c and 3d). This loading arrangement simulates the inertial forces generated during and earthquake and tests the diaphragm-to-frame connections (Figure 2c).

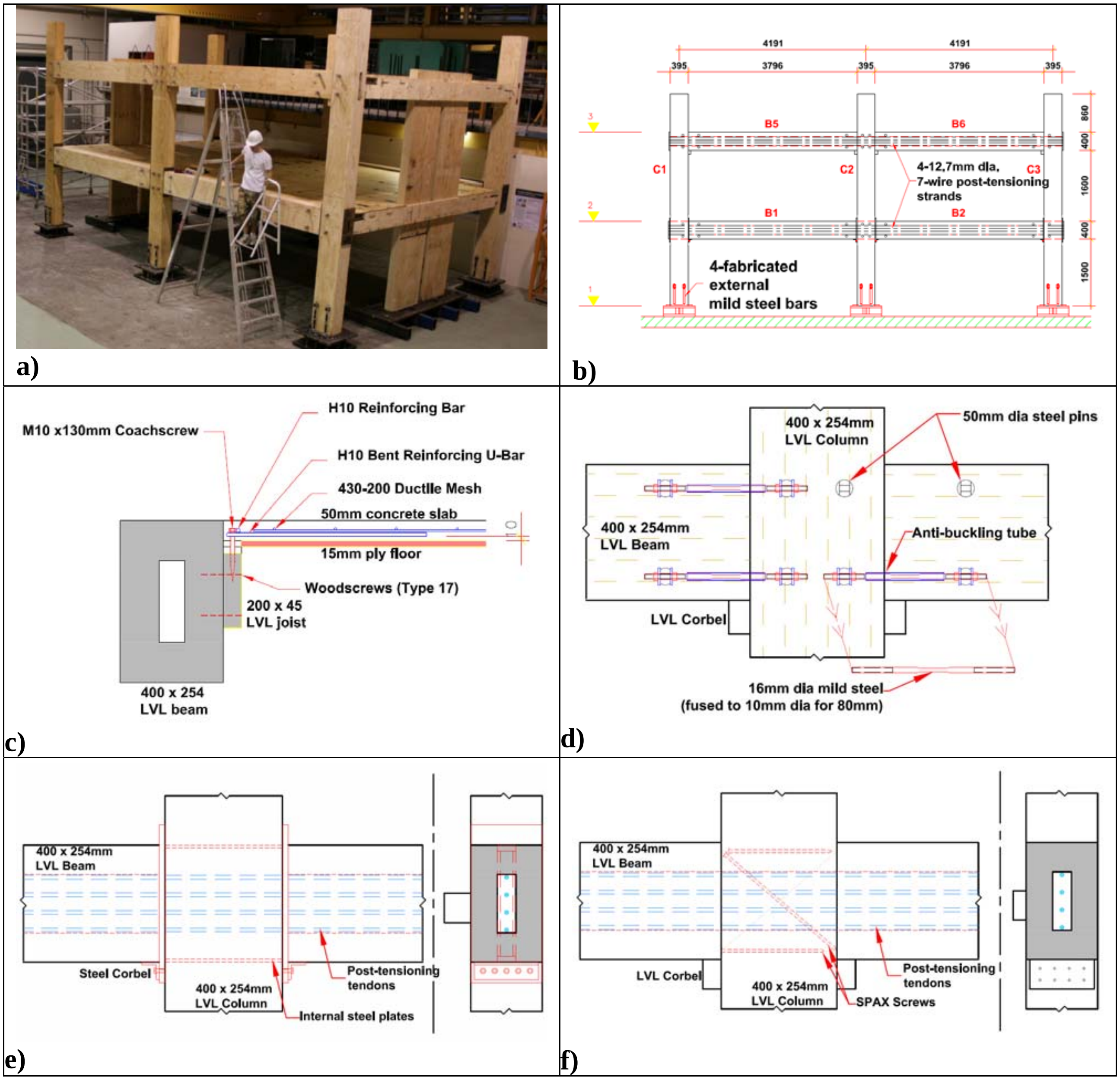

Figure 2. Details of test building; a) 3-D view b) Structural frame elevation c) Concrete slab d) Additional Mild Steel Reinforcement e) Connection detail at Level 2 f) Connection detail at Level 3

Experimental Results

The experimental data from the frames tests are examined to determine the influence of the concrete slab, mild steel reinforcement and column armouring on the seismic response of Pres-Lam frames. The general damage state of the building is also examined.

Lateral Load-Displacement Response

The global hysteretic response of the frames is shown in Figure 4a and b, which plot the total combined base moment for both of the frames versus the top floor drift. Figure 4 demonstrates that the frames remain essentially elastic with little hysteretic energy dissipation (less than 5% equivalent viscous damping) up to the targeted level of displacement. The majority of hysteretic damping is provided from the mild steel reinforcement at the base of the columns, which contributes up to 20% of the total lateral resistance. There is no significant loss in strength or stiffness for repeated cycles.

The Effect of the Concrete Slab

Figure 4a shows that the addition of a concrete slab has limited effect on the lateral resistance of the frames. There are three additional sources of resistance when the diaphragm is present; 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. Taking the total base moment and subtracting the out-of-plan moment resistance of the walls, it is computed that the additional resistance provided by the concrete slab is approximately 15% at 2% drift.

Table 1 shows that there is no apparent increase in the bending moment in the beam-column connections due to slab interaction. In fact, because of the axial load applied to column C1 during Phase 1, the moments are often lower after concrete slab is present. It is likely that the additional resistance is provided by local bending of the floor slab around the exterior columns (discussed further below) and a coupling effect of the floor and walls. In addition, the slab does not affect the initial elastic stiffness of the overall structure.

The Effect of Additional Mild Steel Reinforcement

Figure 4b shows the hysteretic response of the Hybrid and Post-tensioning only (P.T. Only) connections. For the Hybrid connections, at 2% drift the base moment provided by the mild steel was approximately 50% of the moment provided by the post-tensioning.

Up to the design displacement, the additional mild steel reinforcement adopted had little effect on the resistance of the frames. At 2% drift, an additional strength of approximately 10% was provided by the mild steel (which is less than the influence of the concrete slab).

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 at 2% drift. 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. However, these devices will be beneficial to the system response at larger displacements which will be the focus of the next stage of testing.

Furthermore, it is a possibility that using larger diameter mild steel may increase initial stiffness (thus result in earlier activation) as well as higher moment (dissipative) contribution. Parametric studies will be carried out on the numerical model to assess the efficiencies of alternative solutions.

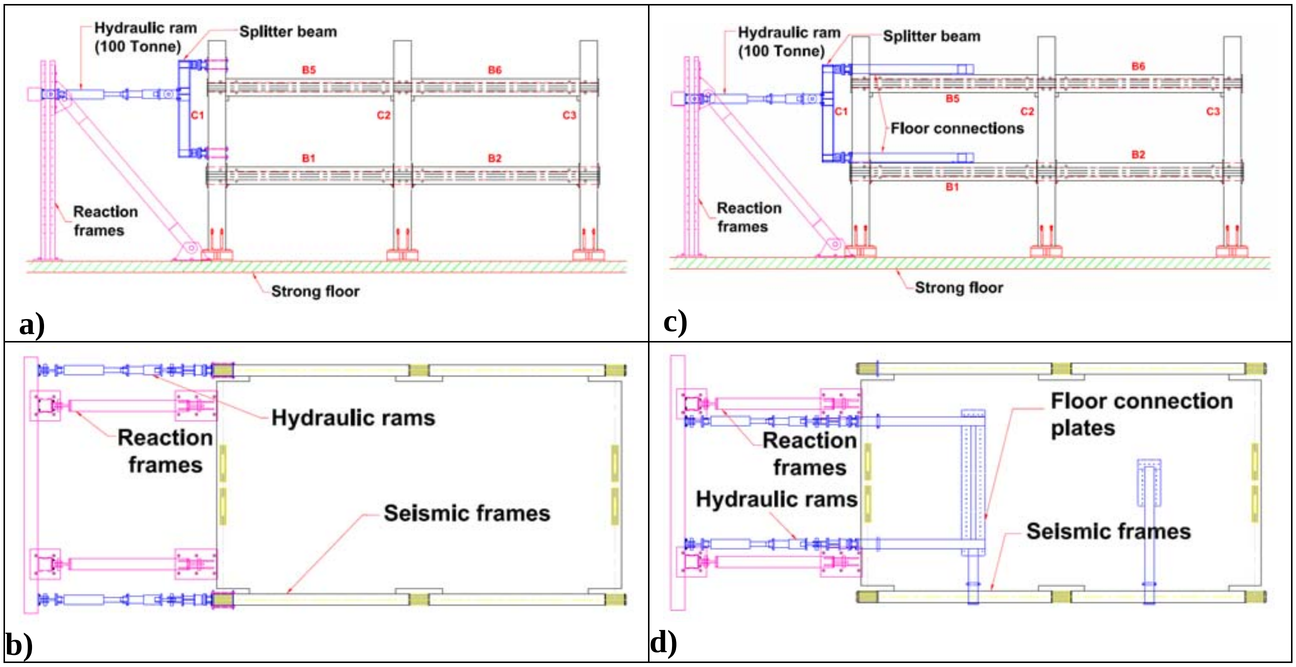

Figure 3. Testing apparatus; a) Phase 1 elevation view b) Phase 1 plan view c) Phase 2 elevation view b) Phase 2 plan view

Connection Response

The influence of joint reinforcement, column face armouring and additional connection reinforcement is examined in terms of the moment-rotation response of the connections. The connection hysteresis for the beam-connections connection on column C3 is given in Figure 4c & d. The peak moment in all beam-column connections is provided in Table 1.

The Effect of Joint Reinforcement

For the post-tensioning only tests the maximum connection rotation is approximately 1%. Hence, the connection deformation is approximately half the total frame deformation. On average, the steel plate reinforced connections (see Figure 2e) on Level 2 have larger rotations than the internal screw-reinforced connections on Level 3. Hence, the steel plate connections appear to be more effective at reducing the elastic deformation of the frame, which is primarily derived from the joint panel region. In addition, from Figure 4c & d it can be observed that the moment demand at Level 3 is roughly half that of Level 2, yet the connection rotation is similar. This suggests that the elastic deformation on Level 3 is much higher, the additional deformation being due to the joint panel deformation suggesting that the screw reinforcement is much less effective than internal steel plates at increasing the joint panel stiffness.

The Effect of Column Armouring

The column armouring used on Level 2 markedly increases the axial stiffness of the beam-column connections. The steel end-plates prevent large perpendicular-to-grain strains. This decreases the neutral axis within the connection which increases the moment capacity.

As mentioned previously, for column C3 the moment at Level 2 is roughly twice that of Level 3. From Table 1, the maximum moment resistance at Level 2 is on average 70% higher than Level 3. These values correspond well with previous analytical research (Newcombe *et al.* 2008a).

The Effect of Mild Steel Reinforcement

The addition of mild steel reinforcement significantly increases the initial stiffness of beam-column connections (see Figure 4c & d). However, both connections types (Level 2 and 3) achieve essentially the same bending moment with and without mild steel reinforcement. It is evident from Figure 4c & d that the connection rotation significantly reduces with the addition of mild steel reinforcement. Yet, the same overall frame displacement is imposed (2% drift). This suggests that mild steel reinforcement induces larger elastic deformation over the frame, again predominately from the joint panel region. This clarifies why there is little difference in the global lateral load response when mild steel is added (see Figure 4b).

Damage Observations

Previous research in small scale subassembly tests without structural diaphragms has shown that Pres-Lam frame connections exhibit very minor levels of damage under large seismic deformation (Palermo *et al.* 2006).

Beam-Column Connections

In general, the timber surrounding all beam-column connections remained elastic up to the design level rotation of 2% Drift (see Figure 5a and b). For phase 1 testing (without a concrete slab), minor damage was induced at the beam-column connections adjacent to column C1. This was caused by the additional axial loads that were applied to the connection via the testing apparatus. The effects of this increased axial load can be seen in Table 1, were there is a significant increase in the connection moment capacity.

Top hung floor system

Top hung floor units were designed with a gap between the timber joist and supporting members to allow seismic rotations (see Figure 5c and d), thus accounting for displacement compatibility of the system as a whole The joist hangers consisted of a cantilevered steel plate fastened to the top of the floor joists (Carradine et al. 2009). This detail avoided any damage to the timber joists and supporting members.

Floor Slab

Minor cracking occurred in the concrete slab during Phase 2 of testing (see Figure 5e and f). No cracking due to geometrical beam elongation of the frames was evident. Localized cracking was observed adjacent to out-of-plane walls due to displacement incompatibility; as the walls rotated the floor adjacent beams remained flat. 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, concentrated around the walls.

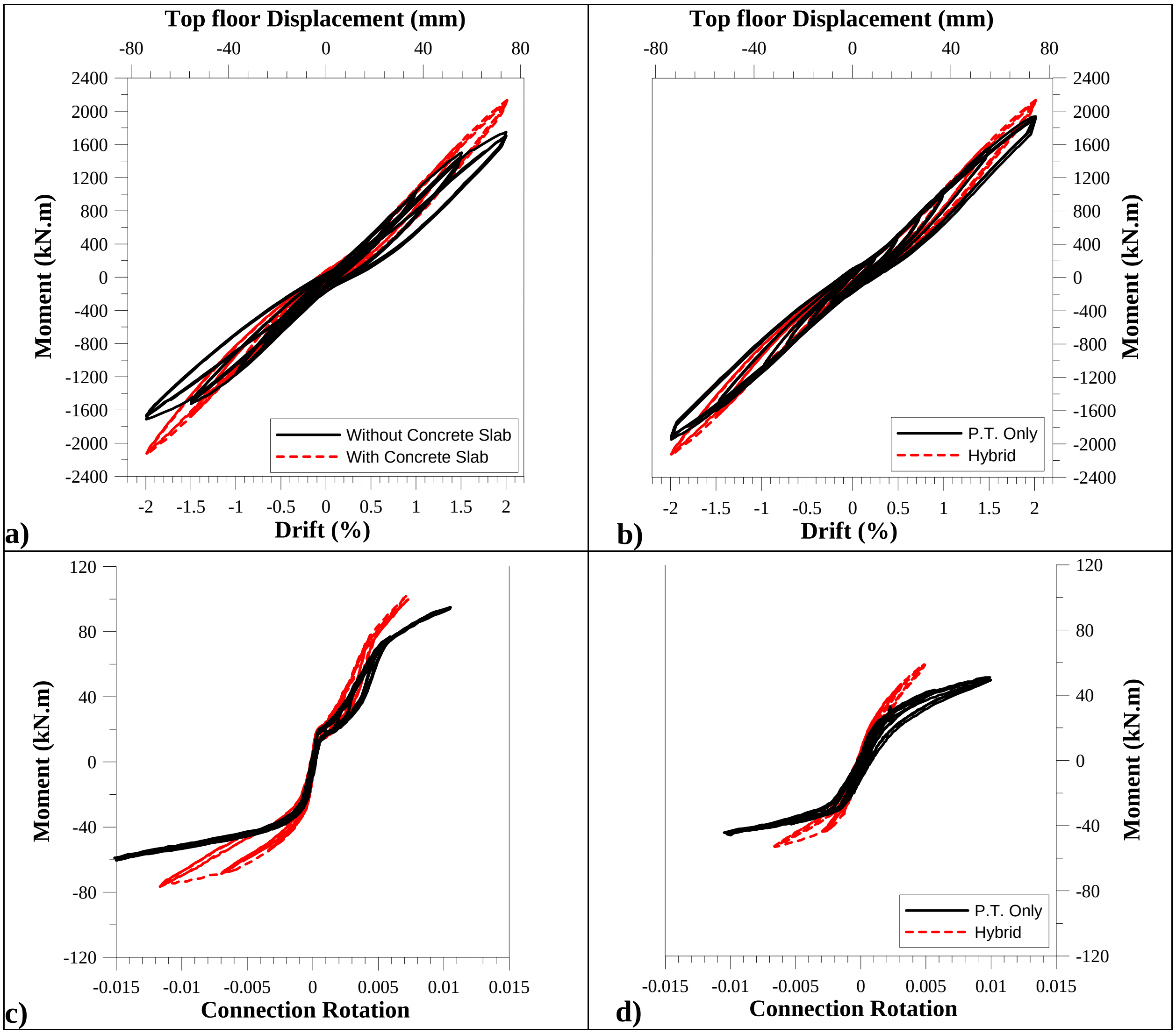

Figure 4. Hysteretic response of the frame; a) Global system hysteresis with and without a concrete slab b) Global system hysteresis with and without additional mild steel c) Moment rotation response of beam-connection C3-B2 (Level 2) d) Moment rotation response of beam-connection C3-B6 (Level 3)

Table 1. Peak moment (kN.m) for each joint in the frame (refer to Figure 2b for beam and column numbers)

		Connection							
Diaphragm	**Reinforcement**	BC*, C1-B1	BC, C1-B5	BC, C2-B1	BC, C2-B5	BC, C2-B2	BC, C2-B6	BC, C3-B2	BC, C3-B6
None (Phase 1)	P.T. Only	72.9	54	68.5	56.3	91.2	49.7	102	52.1
	Hybrid	90.4	71.4	80.6	66	95.5	55.8	102.5	65.3
Concrete slab (Phase 2)	P.T. Only	68.3	49.8	68.3	49.6	92.2	43.1	94.2	50.9
	Hybrid	83.5	47.9	77.1	49.6	95.8	47.1	102.2	59

* BC = Beam column connection

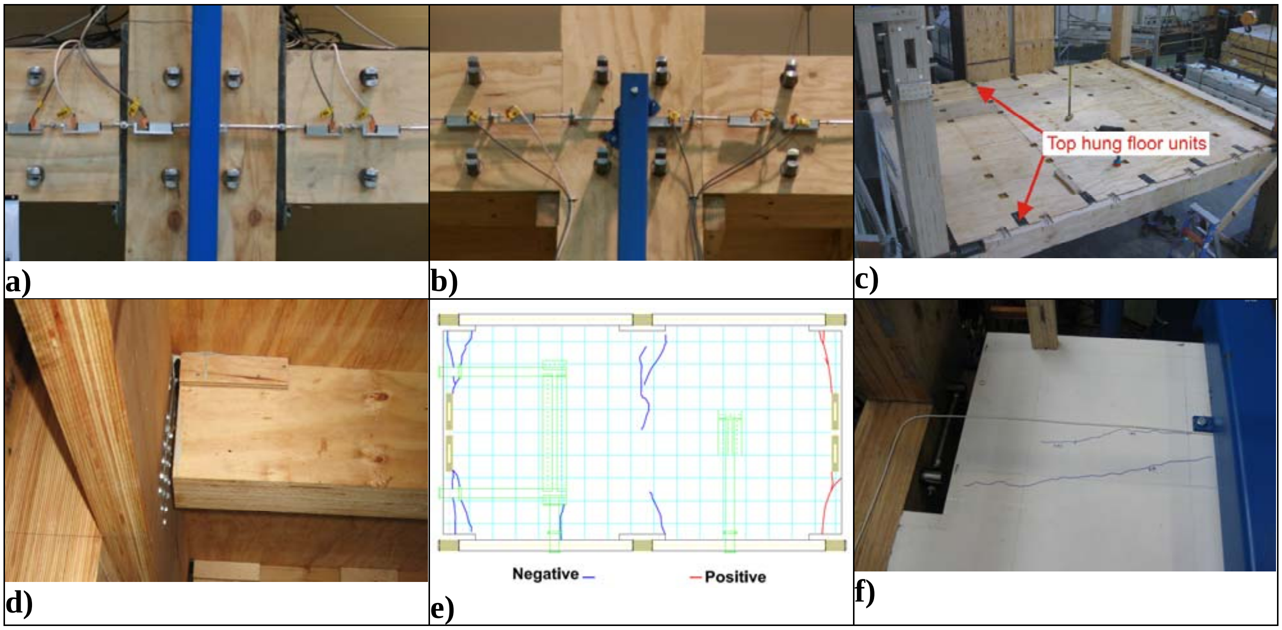

Figure 5. Damage observations; a) Level 2 beam-column joints at 2% drift b) Level 3 beam-column joints at 2% drift c) Top hung floor system floor units (excl. concrete topping) d) Rotation of floor units relative to seismic frames e) Crack pattern of floor slab f) Rotation of edge beams

Conclusions

- Preliminary experimental tests on a two-storey post-tensioned timber frame building gave excellent seismic performance, with almost no damage at 2% drift and full re-centering.
- Elastic deformation of the two-storey frame represented a much larger proportion of total displacement than would occur in a similar reinforced concrete frame.
- The largest component of elastic displacement was shear deformation of the beam-column joint zone. Internal steel plates were more effective than screw reinforcement for reducing this shear deformation.
- Column face armoring with steel plates enhanced the stiffness of the beam column connections by 70% compared with a bare timber.
- The high proportion of elastic deformation 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 expected at larger displacements.
- Compressive deformation of the timber beam-connections limited the overall frame elongation, which in turn limited the increase in flexural stiffness of the frames due to the participation of the floor slab, to approximately 15%. In comparison to traditional concrete systems, this is a marginal increase which can easily be accounted for in design.

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

Buchanan, A., Deam, B., Fragiacomo, M., Pampanin, S., and Palermo, A. 2008. Multi-Storey Prestressed Timber Buildings in New Zealand. *Structural Engineering International, Journal of the International Association for Bridge and Structural Engineering (IABSE)*, Vol. 18(2).

Buchanan, A. H., and Fairweather, R. H. 1993. Seismic Design of Glulam Structures. *Bulletin NZSEE*, Vol. 26(4), pp 415-436.

Carradine, D. M., Newcombe, M. P., and Buchanan, A. H. 2009. Using Screws for Structural Applications in Laminated Veneer Lumber. *CIB-W18 Conference Proceedings*, Dubendorf, Switerland, Paper 42-7-7.

Christopoulos, C., and Pampanin, S. 2004. Towards performance-based design of MDOF structures with explicit consideration on residual deformations. *ISET Journal of Structural Engineering*, Vol. Special Issue on "Performance-Based Seismic Design", Paper 440.

Newcombe, M., van-Beerschoten, W. A., Carradine, D., Pampanin, S., Buchanan, A. H., Deam, B. L., and Fragiacomo, M. 2009. In-plane experimental testing of timber-concrete composite floor diaphragms. *New Zealand Society of Earthquake Engineering Conference*, Chrictchurch, New Zealand.

Newcombe, M. P., Pampanin, S., Buchanan, A., and Palermo, A. 2008a. Section Analysis and Cyclic Behavior of Post-Tensioned Jointed Ductile Connections for Multi-Storey Timber Buildings. *Journal of Earthquake Engineering*, Vol. 12(Special Edition), pp. 83-110.

Newcombe, M. P., Pampanin, S., Buchanan, A., and Palermo, A. 2008b. Seismic design of post-tensioned timber frames. *14th World Conference in Earthquake Engineering*, Beijing, China, pp. 8, Paper No. S12-008.

NZS3101:2006. *Concrete Structures Standard: Appendix B: Special Provisions for Seismic Design of Ductile Jointed Precast Concrete Structural Systems*, New Zealand Standards, Wellington.

Palermo, A., Pampanin, S., and Buchanan, A. 2006. Experimental investigations on LVL seismic resistant wall and frame subassemblies. *First European Conference in Seismic Engineering and Seismology, Geneva, Switzerland*paper 983, pp. 10.

Palermo, A., Pampanin, S., Buchanan, A., and Newcombe, M. 2005. Seismic Design of Multi-Storey Buildings using Laminated Veneer Lumber (LVL). *2005 New Zealand Society of Earthquake Engineering Conference*, Wairaki, New Zealand, pp. 8.

Pampanin, S. 2005. Emerging solutions for High Seismic Performance of Precast/Prestressed Concrete Buildings *Journal of Advanced Concrete Technology, "High Performance Systems"* Vol. 3(2), pp. 202-223.

Peng, B. H. H., Dhakal, R., and Fenwick, R. 2006. Causes of Elongation in Reinforced Concrete Beams Subject to Cyclic loading. *The Tenth Asia-Pacific Conference on Structural Engineering & Construction*, Bangkok, Thailand.

Priestley, M. J. N., Calvi, G. M., and Kowalsky, M. J. [2007]. *Displacement-Based Seismic Design of Structures*, IUSS PRESS, Pavia, Italy.

Priestley, M. J. N., Sritharan, S., Conley, J. R., and Pampanin, S. 1999. Preliminary Results and Conclusions from the PRESSS Five-story Precast Concrete Test-Building. *PCI Journal*, Vol. 44(6), pp. 42-67.

Stanton, J. F., Stone, W. C., and Cheok, G. S. 1997. A Hybrid Reinforced Precast Frame for Seismic Regions. *PCI Journal*, Vol. 42(2), pp. 20-32.