

# Seismic Behaviour of Prestressed Timber Columns under Bi-directional Loading

Asif Iqbal  
Stefano Pampanin  
Andy Buchanan  
Department of Civil Engineering  
University of Canterbury, Christchurch, New Zealand

## Summary

Structural members made of laminated veneer lumber (LVL) in combination with unbonded post-tensioning have recently been proposed, which makes it possible to design moment-resisting frames with longer spans for multi-storey timber buildings. Moreover, prefabricated and prestressed timber buildings can be designed to have enhanced re-centering and energy dissipation characteristics for seismic resistance. The post-tensioning provides re-centering capacity while energy is dissipated through the addition of special dissipating devices.

As part of a research program on timber structures, experimental and analytical studies have been performed to investigate response of prestressed LVL columns under bi-directional seismic loading. The results show excellent seismic performance, characterized by negligible damage of the structural members and small residual deformations, even under the combined effect of loading in two directions. Energy is dissipated mostly through yielding of external mild steel axial dissipators connecting the column and the foundation, which can be easily removed and replaced after an earthquake. Since post-tensioning can be economically performed on site, the system can be easily implemented in multi-storey timber buildings.

## 1. Introduction

Current seismic design philosophies for multi-storey buildings emphasize the importance of designing ductile structural systems which undergo cycles of inelastic displacement during earthquakes, resulting in some residual damage but no significant reduction in strength. Innovative solutions have been developed under the U.S. PRESSS (PREcast Structural Seismic Systems) programme coordinated by the University of California, San Diego [1] for the seismic design of multi-storey precast concrete buildings. Such solutions are based on joints between pre-fabricated elements with unbonded post-tensioning. As a result, efficient structural systems are obtained, which can undergo large inelastic displacements, while limiting the damage to the structural system and assuring full re-centring capability after the seismic event. A particularly efficient solution is provided by the “hybrid” system [2] (Fig. 1) where an appropriate combination of self-centring capacity (unbonded tendons plus axial load) and energy dissipation (mild steel dissipation devices) leads to a “controlled rocking motion”, characterized by a “flag-shaped” hysteresis loop (Fig. 2).

The hybrid concept has been later implemented recently in timber structures [3]. For multi-storey timber construction, LVL, fabricated from sheets of veneer glued into panels, is a suitable material since it has a higher level of homogeneity and superior strength than rough sawn or glulam timber. As part of a comprehensive research investigation for the development of innovative seismic resisting systems for multi-storey timber construction, a number of different frame and wall systems have been successfully tested [4] to [7] under uni-directional loading. In this contribution, timber column-to-foundation subassembly has been tested in three dimensions.

In a practical building frame for multi-storey building, the columns, especially the corner columns, are often subject to displacements in two directions simultaneously. Therefore, it was necessary to test columns under bi-directional loading. A series of tests on cantilever timber columns connected to steel foundation have been carried out. Both post-tensioned only solutions and hybrid solutions with external dissipators are investigated for the specimen. Experimental results for the column-to-foundation subassembly under bidirectional cyclic quasi-static and pseudo-dynamic loading are presented here. Further results of the experimental investigation are presented in Iqbal et al. [8]. The results are discussed here to evaluate the performance of the hybrid connections. Comparisons have been made between results from the experiments and an analytical model.

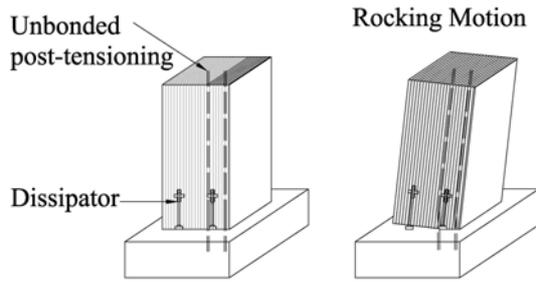


Fig. 1 Hybrid connection and rocking motion

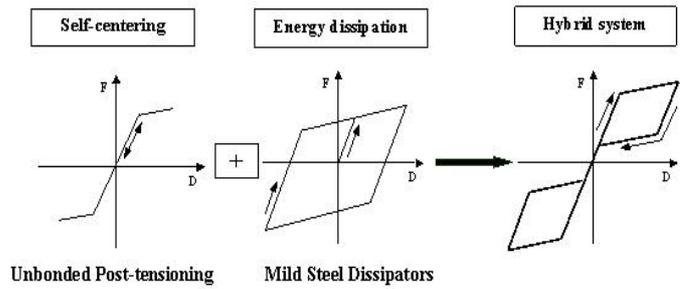


Fig. 2 Flag-shaped hysteresis behaviour[9]

## 2. Experimental Investigation

### 2.1 Properties of the Specimen Tested

The LVL used for the column is Hy90 [10], manufactured in accordance with AS/NZS 4357. For limit states design to the New Zealand Code NZS 3603 [11], Hy90 characteristic strengths are given in Table 1.

Table 1. Material Properties of Hy90 LVL [9]

Modulus of Elasticity Parallel to Grain	E	9000	MPa
Bending Strength	f <sub>b</sub>	35	MPa
Tension Strength Parallel to Grain	f <sub>t</sub>	19	MPa
Compression Strength Parallel to Grain	f <sub>c</sub>	28	MPa
Compression Strength Perpendicular to Grain	f <sub>p</sub>	10	MPa

The test specimen was originally designed as a timber bridge pier to have the moment capacity close to that of a concrete bridge pier tested as part of a recent research project at University of Canterbury, though it can be argued that it was representative of a column from a multi-storey timber building. The column has been constructed to have a hollow section by gluing together four Hy90 sections from standard beam sections, each with a width of 360mm and thickness of 90mm (Fig. 3) to make a column 450mm square. The hollow timber column could be upgraded to higher axial load capacity for high-rise building structures by either making the cavity smaller or adding high strength concrete infill while maintaining the unbonded post-tensioning arrangement. For commercial production it will be preferable to use the arrangement shown in Fig. 3(b) for a large cavity or Fig. 3(c) for a small cavity, because both of these can be manufactured in a standard press.

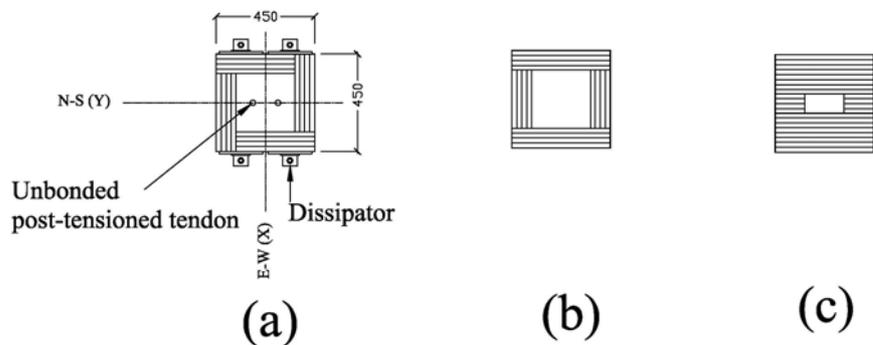


Fig. 3: Column section details

## 2.2 Quasi-static Cyclic Testing

A series of tests were carried out on a single column with alternate arrangements subjected to bi-directional loading. There was no additional axial load applied, and the initial post-tensioning of 50% of tendon yield forces in the two tendons is designed to include some the axial force due to gravity load. The bottom end of the timber column was placed directly on the steel foundation. No attempt was made to increase the bearing strength of the bottom end of the column. The post-tensioning tendons were anchored in a steel plate at the top of the column, and under the steel foundation at the bottom. There was no other contact between the tendons and the column. The cantilever column was horizontally loaded at the expected point of contra-flexure within a frame system, i.e. the mid-level of the inter-storey height (Fig. 4a). The quasi-static loading protocol (Fig. 4b) consists of three cloverleaf-shaped cycles of increasing inter-storey drift, following the acceptance criteria for moment-frames proposed by the ACI T1.1-01, ACI T1.1R-01 [12]. The load was applied simultaneously from two orthogonal directions through hydraulic actuators (Fig. 4c).

Mild steel energy dissipators (Fig. 4d) were added to the column for the hybrid tests. The energy dissipators consist of steel rods designed to yield in both tension and in compression. The 8mm diameter rods are encased in steel tubes injected with epoxy to prevent buckling in compression. The top end of each external dissipator was connected to an external steel case fixed to the LVL column, and the bottom end was fixed to the steel foundation.

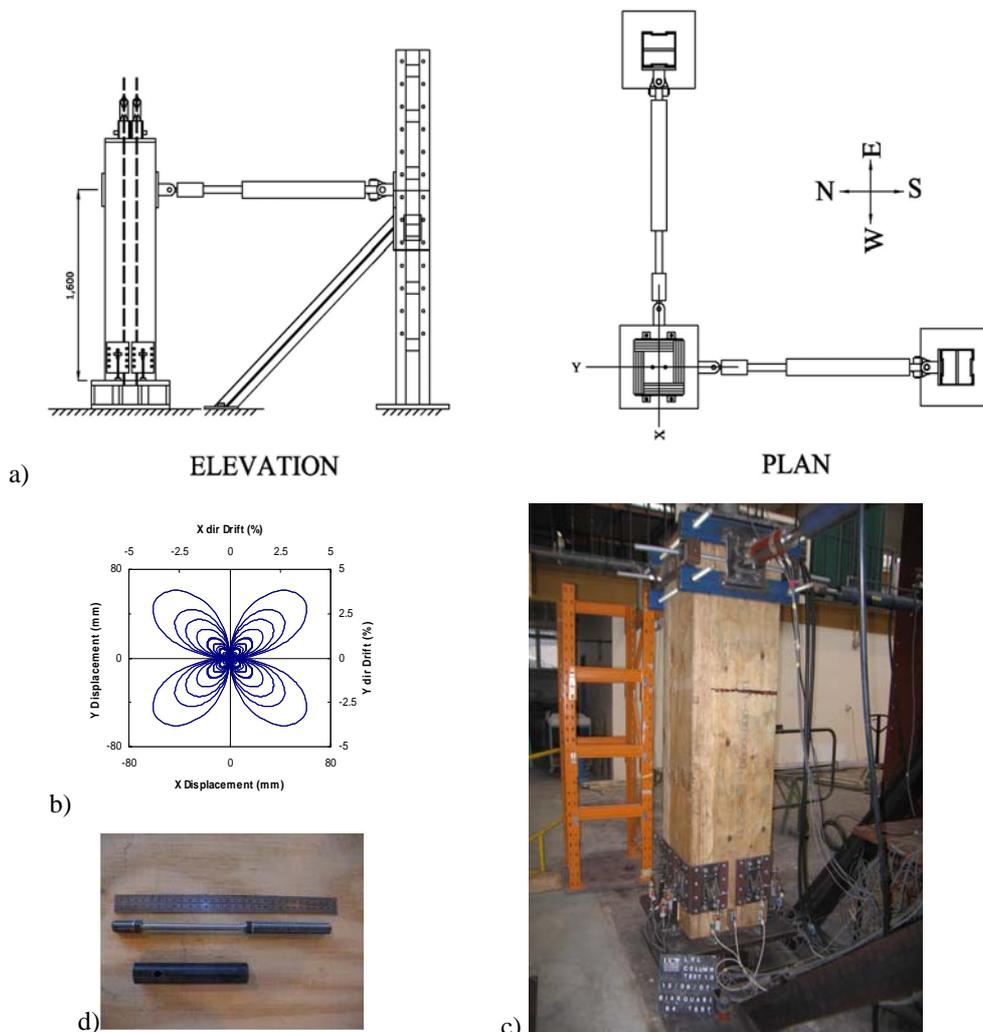


Figure 4: a) Test setup; b) loading protocol; c) testing arrangement; d) dissipators

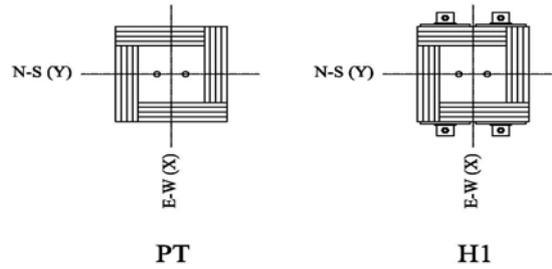


Figure 5: Details of specimens with designations

Two specimens with different levels of dissipation capacity (Fig. 5) were tested. First specimen to be tested was specimen PT with unbonded post-tensioning only and no energy dissipators. The gray lines on Fig. 6(a) and Fig. 6(b) illustrate the recorded values of lateral force vs. drift in the N-S and E-W directions respectively. The tendon force vs. drift is shown in Fig. 7(a). It is visible that there is some energy dissipation due to small inelastic deformations at the base of the column.

The hybrid solution was investigated using the Specimen H1 with two external dissipators placed at each side along N-S direction, parallel to the plane of the tendons (Fig 5.). The black lines in Fig. 6 illustrate the lateral force vs. drift. The tendon force vs. drift is shown in Fig. 7(b). Significant hysteretic dissipation is observed due to yielding of the energy dissipators. It is also important to notice that greater dissipation is achieved in E-W direction, in the plane perpendicular to the tendons, but it also tends to get some residual displacements because of smaller recentering forces from the tendons in that plane. On the other hand, in the direction parallel to the plane of the tendons (N-S) full recentering is achieved due to higher recentering forces from the tendons.

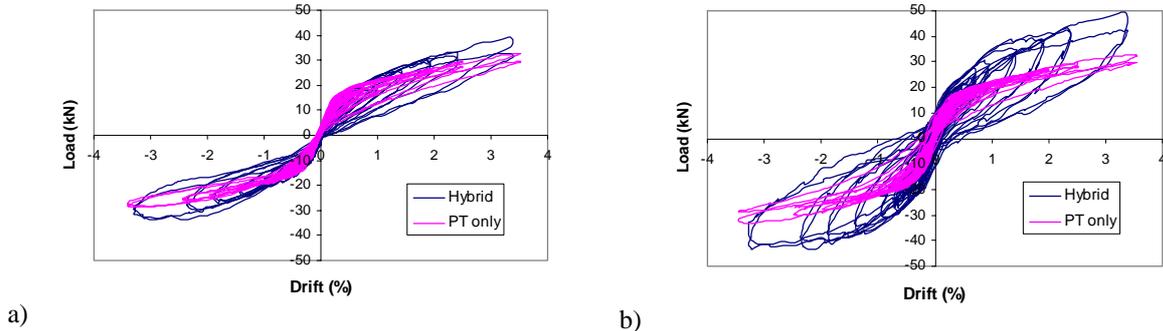


Fig. 6: Load-displacement plots of Specimens PT and H1 a) N-S direction; b) E-W direction

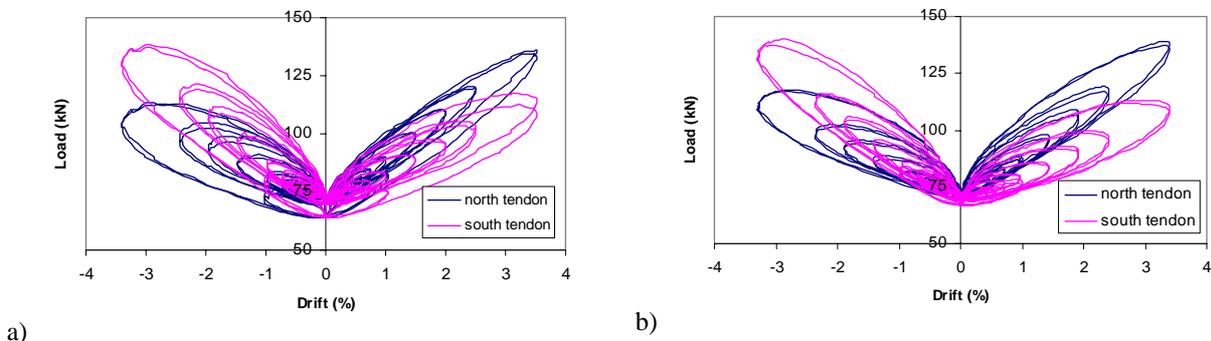


Fig. 7: Plots of tendon forces vs. drift of specimens a) PT and; b) H1

### 2.3 Pseudo-dynamic Testing

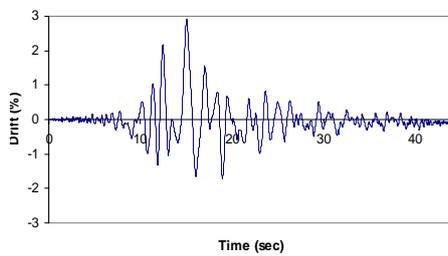
A series of pseudo-dynamic tests was carried out to simulate slow motion dynamic response of the

system subjected to earthquake input ground motions, in both post-tensioned only and hybrid configurations. As part of the required information to solve the equation of motion of the SDOF system within the pseudo-dynamic algorithm, an equivalent mass of 4500 kg was assumed, corresponding to the expected gravity load (dead load plus a portion of the live load) for the tributary area of a column within a single storey timber building. An equivalent viscous damping of 5%, proportional to the initial stiffness was adopted.

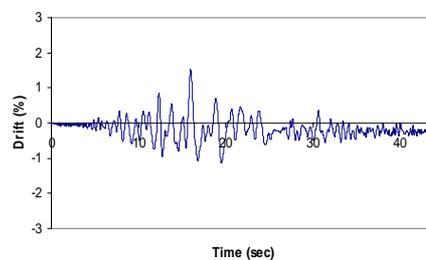
The details of the earthquake ground motions used in the tests are given in Table 2. The test of Specimen ST could not be continued for the whole duration of Landers accelerogram because the maximum drift exceeded the displacement limit of the testing arrangement. Specimen H1 was subjected to a 50% higher intensity of the same earthquake record in order to assure adequate inelastic response and re-centring capability. The response is shown in Figure 8. In spite of the higher intensity of the ground motion, maximum drift of Specimen H1 was less than that of Specimen PT, due to the additional strength and dissipation contribution provided by the external dissipators. A small residual displacement is observed in one direction due to the smaller out-of-plane recentering capacity of the two prestressing tendons.

Table 2. Characteristics of the adopted earthquake events

Event	Year	Mw	Station	Duration, sec	Scaling Factor	Component	PGA, g (scaled)
Landers	1992	7.3	Yermo Fire Station	44.0	2.2	360	0.334
						270	0.245
Cape Mendocino	1992	7.1	Fortuna Blvd	44.0	3.8	000	0.441
						090	0.433

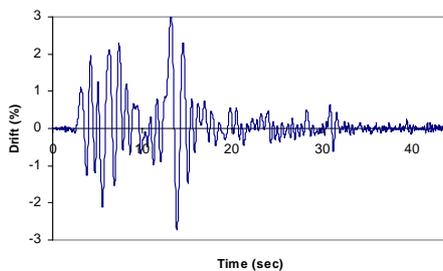


a) N-S direction

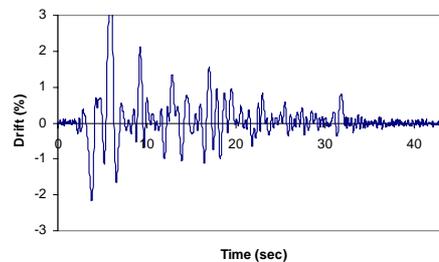


b) E-W direction

Fig. 8 Time history plots of Specimen H1 subject to Landers ground motion



a) N-S direction



a) E-W direction

Fig. 9 Time history plots of Specimen PT subject to Cape Mendocino ground motion

The Specimen PT was tested under Cape Mendocino accelerogram scaled to have intensity

comparable to the Landers earthquake (Table 2). Figure 9 shows the response of Specimen PT in terms of drift time-history. As expected, the maximum drift in this case is greater than that of Specimen H1, but full recentering is achieved despite partial asymmetry of the response.

### 3. Analytical Study

It is necessary to develop analytical models of these solutions for practical structures. A preliminary numerical model of one column-to-foundation system is described below. A simple analytical approach based on section analysis concept and lumped plasticity model [13] is followed to calculate the bi-directional behaviour of the column. Although the procedure was developed for precast concrete structures, it has been found applicable to timber structures with minor adjustments [14].

The lumped plasticity model can be efficiently adopted for hybrid connections where the main inelastic demand is accommodated within discrete critical sections (i.e. at the column-foundation interface). Due to the opening and closing of a single crack at the interface, an infinite curvature is developed at the critical section: therefore a moment-rotation relationship has to be preferred to a traditional moment-curvature when characterizing the section behaviour. Rotational inelastic springs in parallel, with appropriate hysteretic behaviour, are assigned to represent the inelastic action at the column-foundation interface while elastic elements are used to represent the structural members. One rotational spring is assigned a nonlinear elastic rule to represent the self-centring contribution, while for the second spring a hysteresis rule representing the energy dissipation contribution is adopted.

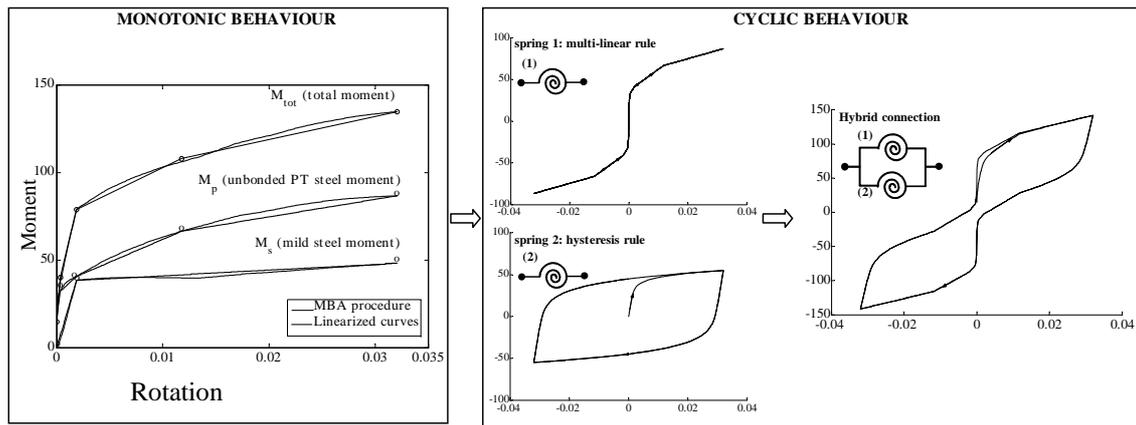


Fig. 9 Calibration of springs referring to Monotonic Beam Analogy [13]

The calibration of the two rotational springs can be obtained by evaluating the monotonic moment-rotation behaviour of each contribution, i.e. mild steel energy dissipation devices and post-tensioning tendons, referring to the Monolithic Beam Analogy (MBA) procedure originally proposed by Pampanin et al. [15] and subsequently refined by Palermo [16], which relies on member compatibility in terms of displacements between a monolithic and a hybrid solution. As represented in Fig. 9, each curve contribution obtained adopting the MBA can be linearized referring to the fundamental performance levels, i.e. the decompression point, loss of linearity point, yielding, serviceability and failure point. Fig. 9 summarizes the above mentioned calibration procedure assuming a Ramberg-Osgood [17] hysteresis rule for the cyclic behavior of dissipator.

The computational scheme is implemented with the Ruaumoko [18] finite-element code. A three-dimensional model of the column and its base connection was created to apply the bi-directional loading. Bi-linear elastic elements have been used to model the post-tensioning tendons while modified Takeda [19] hysteresis was used for the energy dissipators. Fig. 10 and Fig. 11 show the comparison between the analytical and experimental results for post-tensioned only and hybrid solutions respectively.

In general, satisfactory confirmation of the numerical procedure is established from the plots. The dispersions from the experimental results can be attributed to a number of reasons. The column was represented in the model by an elastic element. But in reality, as mentioned earlier, some inelastic deformations take place at the base during the tests leading to greater energy dissipation. Although shear keys were placed around the column base, small amounts of slippage were observed during the tests, which are not accounted for in the numerical model. The timber also goes through strain softening at higher drifts which results in forces lower than computed by the model. Further analytical studies are currently ongoing to provide complete numerical predictions of such systems and these factors can be incorporated in future analytical models currently under development.

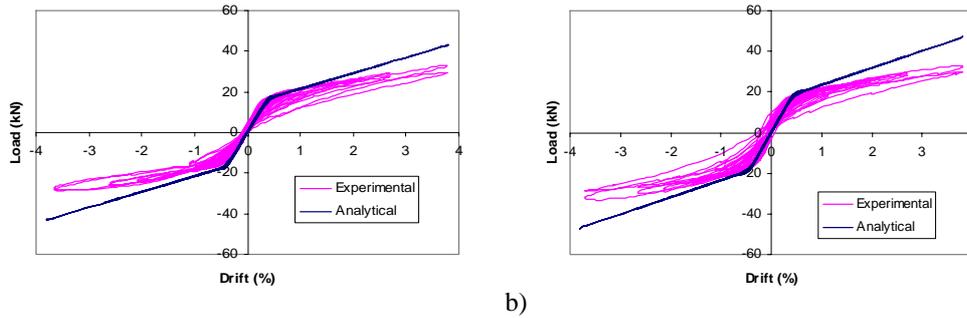


Fig. 10 Comparative plots of PT-only specimen: a) N-S direction; b) E-W direction

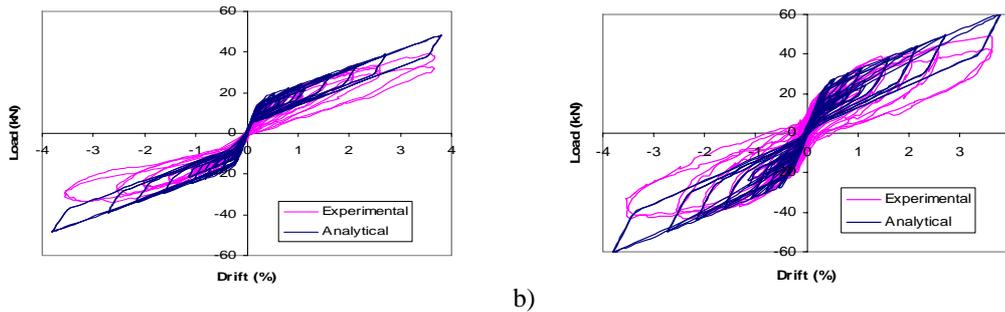


Fig. 11 Comparative plots of Hybrid specimen: a) N-S direction; b) E-W direction

#### 4. Conclusions

The experimental results of cyclic quasi-static and pseudo-dynamic tests on LVL column-to foundation connections under bi-directional quasi-static cyclic and pseudo-dynamic loading further confirmed the applicability of multi-storey timber buildings with hybrid connections. The hybrid systems showed a significantly greater level of energy dissipation compared with the post-tensioned only solution. In all cases, considering different simulations of seismic loading, the tested systems exhibited high levels of ductility, negligible residual deformations and no significant damage of the structural elements. Analytical studies show good agreement between the experimental and numerical results indicating that the behavior of the system in practical applications with can be predicted with high level of confidence.

#### 5. Acknowledgements

The research in this paper is supported by FIDA (Forest Industries Development Agenda). The LVL material was supplied by Carter Holt Harvey. The comments and suggestions from Dr. Alessandro Palermo (Politecnico di Milano, Italy) and Dr. Massimo Fragiaco (University of Sassari, Italy), and technical support from Nigel Dixon and Gavin Keats are gratefully acknowledged.

## 6. References

- [1] Priestley M.J.N., Sritharan S., Conley J.R. & Pampanin S., "Preliminary Results and Conclusions from the PRESSS Five-story Precast Concrete Test-building", *PCI Journal*, Vol. 44(6), 1999, pp. 42-67.
- [2] Stanton J.F., Stone W.C. and Cheok G.S., "A Hybrid Reinforced Precast Frame for Seismic Regions", *PCI Journal*, Vol. 42(2), 1997, pp. 20-32.
- [3] Palermo A., Pampanin S., Buchanan A. and Newcombe M., "Seismic Design of Multi-storey Buildings using Laminated Veneer Lumber (LVL)", *Proc. NZSEE Conference*, Wairakei, New Zealand, 2005, CD-ROM.
- [4] Palermo A., Pampanin S., Fragiaco M. Buchanan A. H. and Deam B. L., "Innovative Seismic Solutions for Multi-storey LVL Timber Buildings", *Proc. of 9th World Conference on Timber Engineering*, Portland, Oregon, 2006, CD-ROM.
- [5] Palermo A., Pampanin S. and Buchanan A., "Experimental Investigations on LVL Seismic Resistant Wall and Frame Subassemblies", *Proc. of First European Conference on Earthquake Engineering and Seismology*, Geneva, Switzerland, 2006, CD-ROM.
- [6] Palermo A., Pampanin S., Fragiaco M., Buchanan A.H., Deam B.L., and Pasticier L., "Quasi-static cyclic tests on seismic-resistant beam-to-column and column-to-foundation subassemblies using LVL", *Proc. Australasian Conf on Mech. and Mat.*, Christchurch, 2006.
- [7] Smith T., Ludwig F., Pampanin S., Fragiaco M., Buchanan A., Deam, B., and Palermo, A., "Seismic Response of Hybrid-LVL Coupled Walls under Quasi-static and Pseudo-dynamic Testing", *Proc. NZSEE Conference*, Palmerston North, New Zealand, 2007, CD-ROM.
- [8] Iqbal A., Pampanin S., and Buchanan A.H., "Experimental Study of Prestressed Timber Columns under Bi-directional Seismic Loading", *Proc. NZSEE Conference*, Wairakei, 2008.
- [9] NZS 3101, "Appendix B: Special provisions for the seismic design of ductile jointed precast concrete structural systems", Standards New Zealand, Wellington, New Zealand, 2006.
- [10] Futurebuild, "Engineered Building Products - Information for Design and Installation", Carter Holt Harvey Limited, Auckland, New Zealand, 2003.
- [11] NZS 3603, "Timber Structures Standard", Standards New Zealand, Wellington, 1996.
- [12] ACI T1.1-01 and ACI T1.1R-01, *Acceptance Criteria for Moment Frames Based on Structural Testing (T1.1-01) and commentary (T1.1R-01)*, ACI Innovation Task group 1 and Collaborators, 2006.
- [13] Palermo A., Pampanin S. and Carr A., "Efficiency of Simplified Alternative Modelling Approaches to Predict the Seismic Response of Precast Concrete Hybrid Systems", *Proc. fib Symposium "Keep Concrete Attractive"*, Budapest, Hungary, 2005.
- [14] Newcombe M.P., "Seismic Design of Multi-storey Post-Tensioned Timber Buildings", Dissertation for Masters Degree in Earthquake Engineering, European School for Advanced Studies in Reduction Of Seismic Risk (Rose School), Pavia, Italy, 2007.
- [15] Pampanin S., Priestley M.J.N., and Sritharan S., "Analytical Modelling of the Seismic Behaviour of Precast Concrete Frames Designed with Ductile Connections", *Journal of Earthquake Engineering*, Vol. 5, No. 3, 2001, pp. 329-367.
- [16] Palermo A., "The Use of Controlled Rocking in the Seismic Design of Bridges", Ph.D. thesis, Dept. of Structural Engineering, Technical University of Milan, Italy, 2004.
- [17] Ramberg W. and Osgood. W.R., "Description of Stress-Strain Curves by Three Parameters", National Advisory Committee on Aeronautics, Technical Note 902, 1943.
- [18] Carr A.J., "RUAUMOKO Program for Inelastic Dynamic Analysis – Users Manual", Department of Civil Engineering, University of Canterbury, New Zealand, 2005.
- [19] Otani S., "Hysteresis Models of Reinforced Concrete for Earthquake Response Analysis", *J. Faculty of Engineering*, University of Tokyo, Tokyo, Vol. XXXVI, No. 2, 1981, pp 125-159.