Experimental investigation of wall-to-floor connections in post-tensioned timber buildings

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ABSTRACT: Rocking timber walls provide an excellent lateral load resisting system for structures using the low damage seismic design philosophy. Special attention has to be given to the wall-to-floor connections, because diaphragm forces have to be properly transferred while accommodating displacement incompatibilities, which include the relative rotation and the uplift of the wall with respect to the floor.

This paper presents the experimental behaviour of several different wall-to-floor connections in Pres-Lam post-tensioned timber structures subjected to horizontal seismic loading. A 2/3 scale post-tensioned timber wall was laterally loaded through collector beams using different connection details.

Bolted connections take advantage of the flexibility of the fasteners and lead to some bending of the collector beam, whereas pins and slotted steel plates reduce the wall-to-floor interaction, as they allow for rotation and some uplift. No significant damage to the floors was observed in any of the tests.

The experimental results showed that floor damage can generally be prevented up to high levels of drift by the flexibility of well-designed connections and the flexibility of the collector beams. In the case of very stiff floors or very stiff collector beams, a more sophisticated connection such as sliding steel elements with a vertical slot should be considered.

1 INTRODUCTION

1.1 System description and problem identification

Until recently the requirement for earthquake safe buildings was based on life safety which can be achieved by applying capacity design principles to the ultimate limit state design of structural elements. In addition to human losses in the recent earthquake series in Canterbury, mainly due to the collapse of two reinforced concrete buildings, the economic impact of the event is enormous as most reinforced concrete buildings were severely damaged during the earthquake and many have been demolished. It is clear that the financial losses due to business interruption, repair and rebuilt of buildings cannot be sustained in the future and a new design philosophy to guarantee more resilient structures is necessary. The low damage Pres-Lam technology developed at the University of Canterbury (Palermo et al. 2005; Buchanan et al. 2011) is taking advantage of the combination of locally grown and engineered timber products and the low-damage characteristics of jointed-ductile PRESSS technology (PREcast Seismic Structural System) originally pioneered in the US (Priestley et al. 1999) and further refined in the last decade at the University of Canterbury (Pampanin 2005; NZCS 2010). Post-tensioned frames and walls provide self-centering lateral load resisting systems and special steel elements provide additional dissipation to the structure. The yielding dissipaters are the only elements which might need replacement after a major seismic event.

A comprehensive design guideline for the design of Pres-Lam frame and wall structures is available in the *Post-Tensioned Timber Buildings - Design Guide Australia and New Zealand* (Structural Timber Innovation Company (STIC) 2013). An overview of issues, potential solutions and on-going research effort regarding the displacement incompatibilities of diaphragms in frame structures have been addressed by Moroder et al. (2013). The interaction between floor diaphragms and walls is discussed in this paper. As shown in Figure 1, the floor diaphragms are subjected to out-of-plane bending when the wall starts rocking, because the wall is rotating and uplifting from the foundation. The gap opening at the base is a peculiarity of the rocking system and is used to activate the dissipators; it however creates vertical displacement incompatibilities as the floor diaphragm tends to remain straight. As

Figure 1 is showing, the incompatibility issue is not unique to rocking systems. In a cantilever cast-in-situ wall the rotation and uplift is concentrated along the plastic hinge length (Henry et al. 2012). It is hence necessary to understand how the diaphragm can be connected to the wall to avoid damage to all adjacent structural elements.

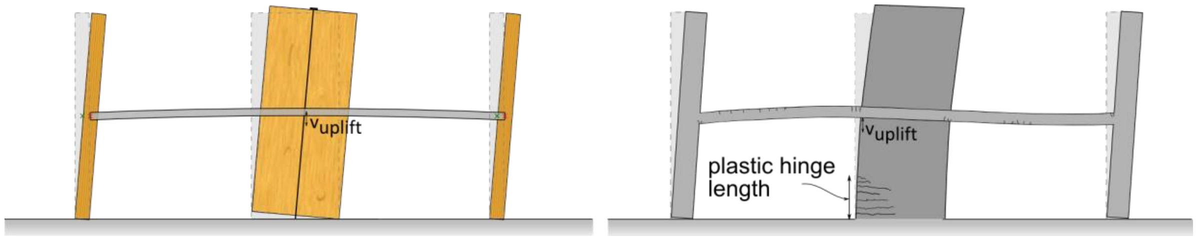


Figure 1. Floor out-of-plane bending due to wall rotation and uplift for a Pres-Lam building and a traditional concrete building

1.2 Related research and built examples

Almost from its original conception, the research on post-tensioned and gravity rocking walls and frames have highlighted the need to consider and resolve displacement incompatibilities of floor diaphragms with the lateral load resisting system. It is worth mentioning that this issue is independent from the type of system (rocking or monolithic) and material used (e.g. Park et al. (2003)). For walls, solutions with slotted holes have been tested successfully in large scale pseudo-dynamic tests or on shaking tables (Priestley et al. 1999; Park et al. 2003; Schoettler et al. 2009). More recently some solutions for steel rocking frames have been proposed by Eatherton (2010), who suggested adding gravity columns and a block-out in the slab to allow rocking. The slab can be connected to the frames by special shear plates or rollers which allow vertical uplift. A case study about possible connections between steel rocking frames and the slab has been given in Latham et al. (2013). For the design of the *Kilmore Medical Centre* in Christchurch a protruding tongue plate which allows for uplift and rotation has been implemented.

Newcombe (2011) studied the response of a Pres-Lam building erected and tested under lateral loading at the University of Canterbury. When the building was loaded in the wall direction, a noticeable increase in the system strength was observed after a timber-concrete-composite floor was added to the structure. The increase of 20% however can be attributed to the fact that the stiff collector beams have been connected to the external edges of the coupled walls. While one edge of a wall was uplifting, the edge of the other wall was immobile because of the compressive force, bending the collector beam over a very short span. This connection geometry must be avoided as damage to the connection or the floor diaphragm is likely to occur.

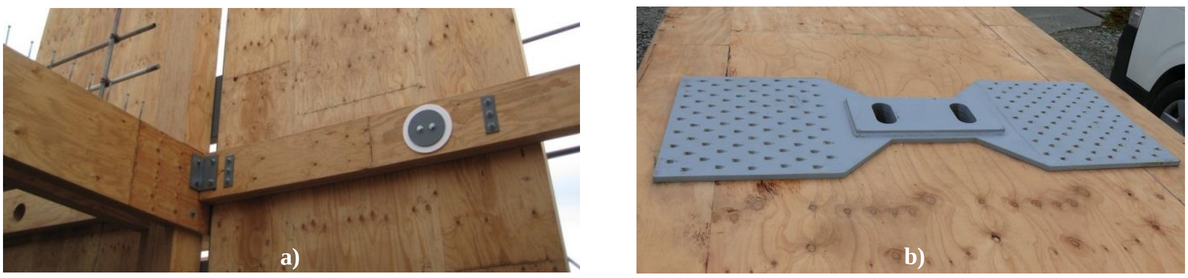


Figure 2. Connection of collector beams to walls. a) NMIT Building in Nelson with a large diameter pin (Devereux et al. 2011); b) Trimble Building in Christchurch with a steel-to-steel connection (Brown et al. 2012)

Another solution was chosen in the design of the *NMIT Arts and Media Building* in Nelson designed by Aurecon (Devereux et al. 2011) where the collector beams have been connected to the coupled walls by 200 mm steel pins, located slightly offset from the centre of the wall to avoid the central post-tensioning cables. The non-continuity of the collector beam and the distance between the pins create little restraint in case of wall uplift. Furthermore, the pin acts as a hinge so that no rotation will be imposed on the beam. A connection design which is supposed to eliminate any displacement incompatibilities has been built in the *Trimble Building* in Christchurch designed by Opus International Consultants (Brown et al. 2012). A steel plate with a vertically slotted hole has been attached to the timber wall; similarly a steel plate with a round hole was attached to the collector beam, so that a simple steel pin can transfer horizontal forces while allowing for uplift and rotation. In the currently under construction *Kaikoura District Council Building* designed by Nelson Timber Solutions Ltd, each timber-to-timber connection between the collector beam and the wall is a group of bolts arranged in a circular pattern. Because of the confined connection geometry, rotation is allowed, uplift of the beam however needs to be accounted for by checking the bending strength and stiffness of the collector beams.

Figure 3 shows a summary of three proposed wall-to-beam connections in timber buildings. In all of these, the rotation incompatibility is accommodated by a single pin or by closely spaced fasteners with small rotational stiffness. Vertical displacements are tolerated by the bending of the collector beam or are eliminated by vertically slotted holes. The figure also shows that if the displacement incompatibilities are not accommodated by a pin and/or slotted holes, the interaction can be accounted by adding rotational and linear stiffnesses to the system.

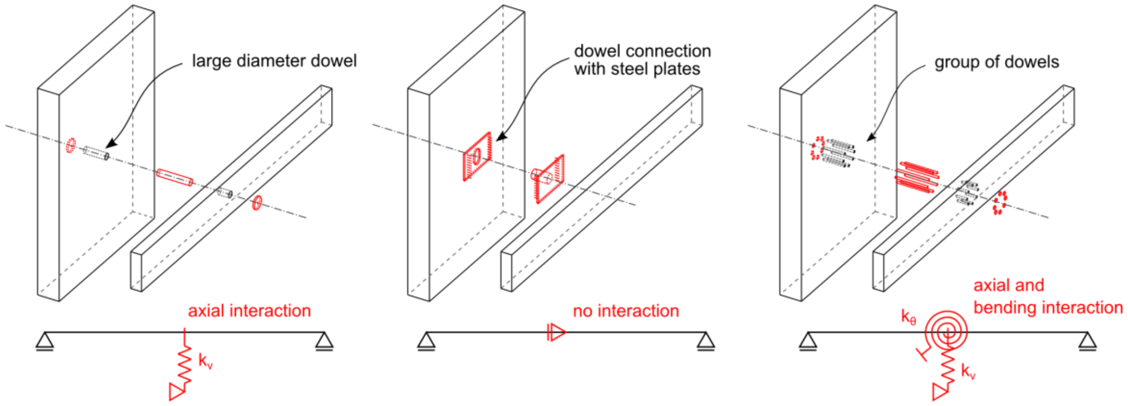


Figure 3. Summary of three proposed wall-to-beam connections with their relative idealized static system of the beam including the connection

The experimental investigation tested the behaviour of several connection details between walls and collector beams, ranging from centred and eccentric bolted connections, external timber blocks, pin connections with and without slots, and a steel comb with slotted holes, to a steel-to-steel connection with a slot. The effective degree of allowance for rotation and uplift will be discussed and design recommendations are given.

This paper describes issues and solutions to address displacement incompatibilities encountered in rocking wall structures. Related research work in steel and concrete structures is summarized and references to recently built examples in timber are made. The test setup of a 2/3 scale laminated veneer lumber (LVL) post-tensioned wall with collector beams is described and several different connection details are introduced. The results of the different connections are then discussed and conclusions drawn. This research is limited to elastic non-dissipative connections; further tests on other kind of connections will be carried out.

2 EXPERIMENTAL TESTING

2.1 Test specimen

The 2/3 scale wall in Figure 4 was initially tested without the timber beams and columns, with details and results given by Sarti et al. (2014). The test specimen was subsequently modified by adding collector beams and gravity columns as shown, so that the wall could be loaded horizontally through the collector beams and the behaviour of the seven different connection configurations between the beams and the wall (described later in the text and shown in Figure 5) could be studied.

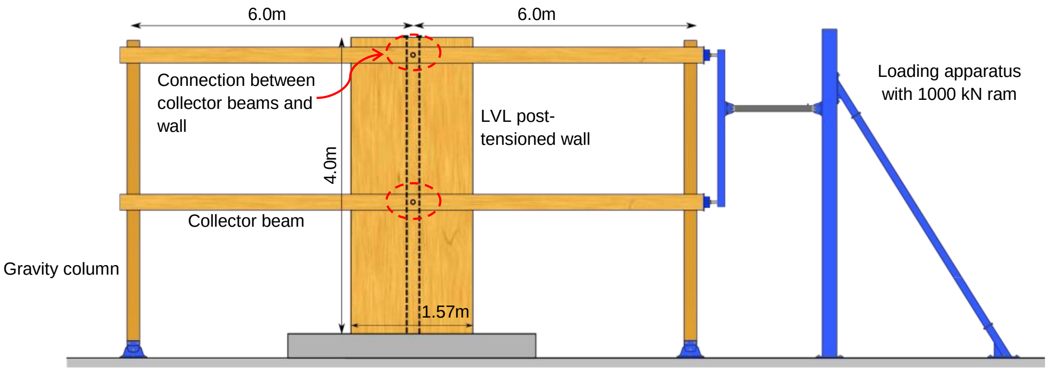


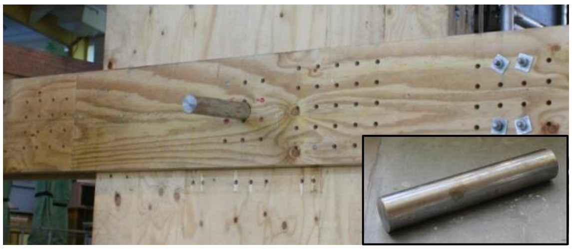
Figure 4. Test setup of the post-tensioned wall loaded through collector beams

The LVL wall was 1570mm wide, 189mm deep and 4m high, and was post-tensioned to the concrete foundation using two 32mm MacAlloy bars with an initial force of 300kN each. The interaction with the diaphragms has been simulated by means of two 6m long collector beams for each floor. These 63x400mm LVL beams have been designed to have the same stiffness as a timber-concrete-composite floor with a 90mm concrete slab connected to two 90x350mm LVL beams with trapezoidal notches. This represents the worst case scenario in terms of beam stiffness, considering a traditional plywood floor, a glued stressed-skin timber floor and a TCC floor.

1) 12 centred 16mm bolts

2) 25 eccentric 16mm bolts

3) timber blocks pushing against the wall



4) centred 65mm diameter pin

5) centred 65 mm diameter pin with slot in wall

6) steel comb with slotted holes

7) steel-to-steel connection with a 40 mm pin in slotted hole

Figure 5. The seven tested connection details between the collector beams and the wall

The collector beams were connected to the gravity columns with four 16mm bolts, which were assumed to work as a hinge to hold the beams down when they were pushed upwards by the rocking wall. The columns were connected to the floor by hinged connections, with load cells to measure axial forces. The horizontal load was applied to the collector beams via a spreader beam, simulating an inverted triangular load.

Seven different types of connection were used to transfer the horizontal load from the collector beams to the wall. As shown in Figure 5, these were:

1) Group of 12 - 16mm bolts at the centre of the wall;
2) Group of 25 - 16mm bolts, 400 mm offset from the centre of the wall;
3) External timber blocks pushing against the edge of the wall, bolted to the beams with 16 bolts;
4) A 65mm diameter round steel pin through a circular hole at the centre of the wall;
5) A 65mm diameter round-square-round steel pin through a vertical slot at the centre of the wall;
6) A steel comb with 7 - 16mm bolts in slotted holes;
7) A steel-to-steel connection with a 40mm diameter round-square-round steel pin in a slotted hole.

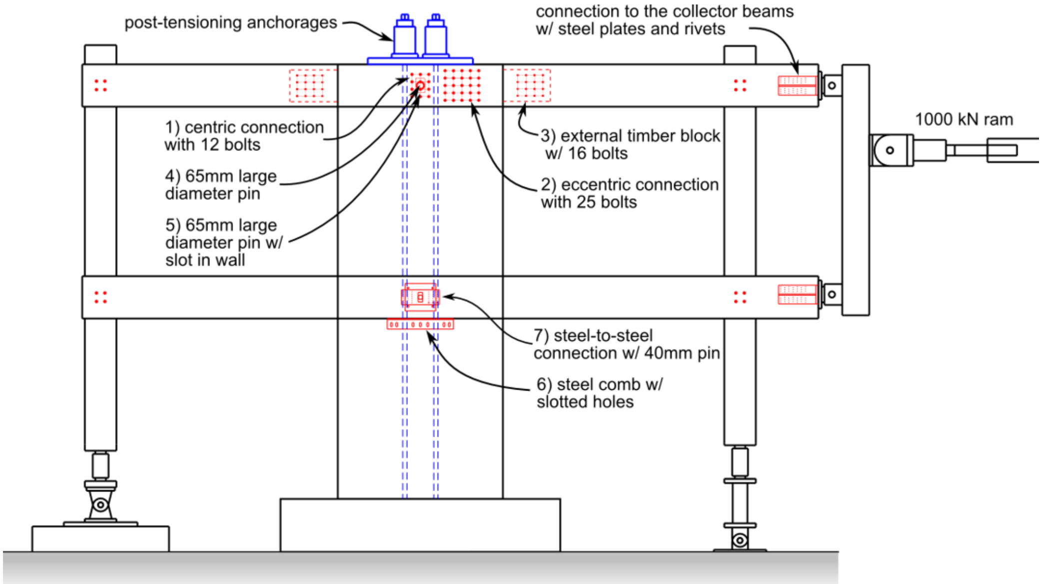


Figure 6. Scheme of connection layouts and positions. Each connection type was tested on the top and bottom beam simultaneously

Because of the length of the bolts, M16 8.8 threaded rods were used for connections type 1 to 3. To avoid splitting, the beams were reinforced with fully threaded screws. The eccentric connection (type 2) was placed 400mm offset from the wall centre to avoid the central location of the post-tensioning bars. Furthermore, depending on the span direction of the floor, gravity forces may need to be transferred through these connections, requiring even more fasteners. For connection type 3, two timber blocks were bolted to the beam, each with a tight fit against one edge of the wall, so that horizontal forces in the beam could be transferred into the wall by compression. The timber blocks were assembled with a high density polyethylene (HDPE) plate and the wall was covered with a stainless steel plate to reduce friction.

Embedment strengths are not available for large diameter steel pins in timber, so values need to be chosen conservatively. For connection type 5, the sliding pin was bearing against two HDPE plates to avoid crushing of the timber perpendicular to the fibres and to reduce friction. Whereas for connection type 4 round pins were used, connection types 5 and 7 had special machined pins with an inner square section where they passed through the wall to avoid stress concentrations on the bearing areas and round sections where they were bearing against the beams. The steel comb consisted in an L-shaped steel profile connected with 65mm long rivets to the beams and seven M16 bolts in slotted holes against the wall. For connection type 7) one steel plate was connected to the beam by 65mm long rivets and the other was fixed to the wall by ZD plates (SWG Schraubenwerk Gaisbach GmbH 2012) with Ø10x280mm fully threaded screws inclined at 30 degrees.

Connections type 1 and type 2 do not act exactly like hinges, but they do allow for a certain degree of rotation because of the confined geometry and flexibility of each single fastener. The remaining connections were designed to not impose any rotation on the beam. The connections types 3, 5, 6 and 7 were designed to allow the wall to move upwards without lifting the beams.

2.2 Test setup and loading protocol

The wall was loaded through two pairs of collector beams following a quasi-static cyclic loading protocol according to ACI ITG-5.1-07 (ACI Innovation Task Group 5 2008). The imposed displacements were applied with increasing amplitudes of three cycles each with corresponding drift values of 0.3%, 0.45%, 0.6%, 1.0%, 1.5% and 2.0%. The displacements were measured directly on the wall, thus avoiding any influence from the slip in the connections or the loading apparatus. The force was applied by a 1000 kN hydraulic ram. Two additional load cells measured the axial force in the gravity columns to evaluate the uplifting forces in the beams. The rotation and uplift of the collector beams were measured with inclinometers and string pots respectively.

3 RESULTS

The results of main interest are the rotation and uplift of the beam relative to the initial position.

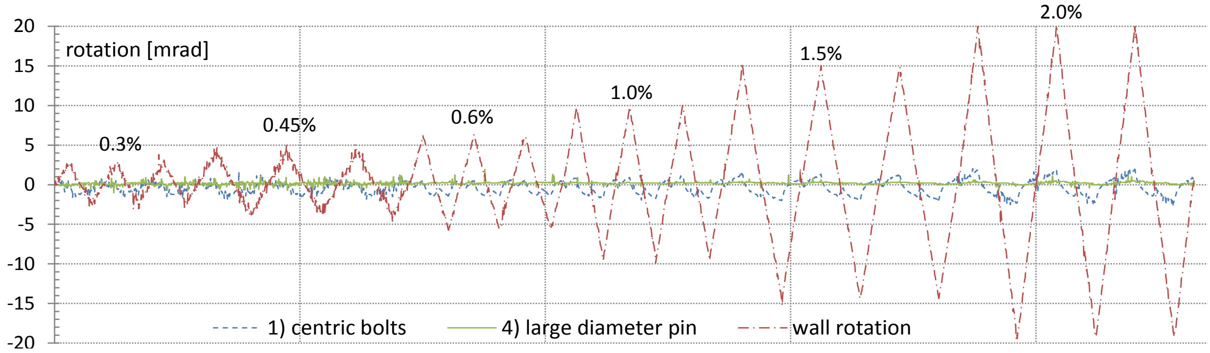


Figure 7. Wall rotation and upper beam rotation for connections type 1 and type 4

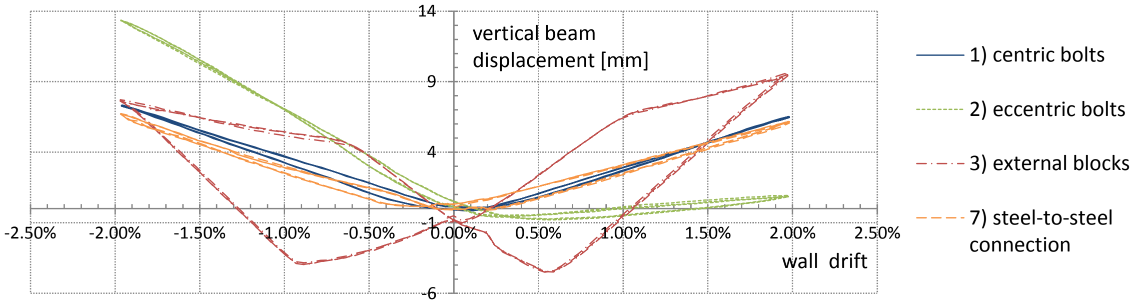


Figure 8. Upper beam uplift at wall centre for the 2.0% drift cycles for connections 1), 2), 3) and 7)

Figure 7 shows the top of wall and upper beam rotations through the full loading protocol. As expected, the beam rotations were much smaller than the wall rotation, because all the connections allowed some differential rotation. As can be seen in Figure 7, the connections with several bolts still forced the beam to follow the wall rotation, but these rotations were very small. On the other hand, connections with large diameter pins allowed the beam to remain straight throughout the testing.

To evaluate the degree of uplift in the collector beams, Figure 8 shows the absolute vertical displacement of the beam plotted against the top wall drift for the 2.0% drift cycles. As the vertical slip in connection type 1 is negligible, the uplift of the beam was equal to the uplift of the wall. The slightly asymmetric behaviour is attributed to the loading apparatus and the different behaviour of the wall when being pushed or pulled. In the case of the eccentric bolted connection (type 2) the uplift at the centre was higher when pulled and almost nil when pushed. The connection with external blocks (type 3) showed a very different behaviour with much higher uplifts than the wall itself. The beam was first being pushed down and then uplifted quite rapidly. When unloaded, the beam lowered back to the initial position with some delay due to friction. Uplift plots for connections type 5 to 6 (not shown) are similar to the plots for connection type 1 and type 7.

Table 1 summarizes the key results for all seven connections, showing the maximum and minimum horizontal forces needed to reach ±2.0% drift and compares them with the values for the same wall tested without diaphragm beams. The 2.0% drift corresponds to a maximum credible earthquake (MCE) design level. When designing for a design basis earthquake (DBE), the drift and therefore forces and displacements are smaller. Table 1 also provides the rotations and uplifts of the upper beam and compares them with the rotation and uplift of the wall itself. Finally the axial forces in the external columns are given; these are good indicators of the influence of the displacement incompatibilities on the system. All values are averages of the maxima during the three cycles at 2.0% drift.

Table 1. Average values for the peak drifts of 2.0% for some key values

Connection		Force on wall		Upper beam		Ext columns	
		- force [kN] [%]	+ force [kN] [%]	rotation [mrad] [%]	uplift [mm] [%]	F_1 [kN]	F_2 [kN]
1)	Group of 12 bolts	-248 (104%)	237 (111%)	2.6 (13%)	7.3 (96%)	16	26
2)	Eccentric group of 25 bolts	-259 (109%)	272 (128%)	2.6 (13%)	13.3 (181%)	12	55
3)	External blocks with dowels	-246 (104%)	248 (117%)	2.3 (11%)	9.5 (131%)	25	32
4)	Large diameter pin	-232 (97%)	227 (107%)	0.9 (5%)	7.4 (97%)	12	14
5)	Large diameter pin and slot in wall	-231 (97%)	230 (108%)	1.0 (5%)	7.9 (107%)	11	17
6)	Steel comb with slots	-251 (106%)	250 (118%)	4.4 (25%)	7.4 (100%)	22	27
7)	Steel-to-steel connection with slots	-231 (97%)	220 (104%)	0.8 (4%)	6.7 (92%)	11	12
		Ratios are relative to the max and min force values of 212kN and -238kN respectively of the wall tested without beams		Ratios are relative to the average wall rotation of 20mrad and wall uplift of 7.5mm respectively		Axial force in the left and right gravity column respectively	

The test series was carried out by using post-tensioning only. In a real wall design mild steel dissipators are normally added, resulting in a flag-shaped hysteresis loop. By changing the ratio between post-tensioning and mild steel $\lambda = M_{PT}/M_S$, the interaction between the wall and the diaphragm has to be accounted accordingly (NZCS 2010; Structural Timber Innovation Company (STIC) 2013).

4 DISCUSSION

Connection type 1 - centred group of 12 bolts:
This connection had a relatively small rotational stiffness and forced the beam to rotate slightly. Because of the centred position and the high linear stiffness, the beam followed the wall uplift closely. The stiffness of the system was increased slightly when compared to the wall tested without beams.

Connection type 2 - eccentric group of 25 bolts:
Because of the eccentric connection, the uplift of the beam was noticeably higher when the wall was pushed. Similarly the strength of the system had a big increase in the same direction, as the beam had to be bend closer to the support. Even with a higher rotational stiffness, the beam did not rotate more than with connection type 1.

Connection type 3 - external blocks with dowels:
Although the rotation of the beam is similar to the bolted connections (type 1 and type 2), the uplift of the beam was bigger, as the timber blocks were pushed up by the wall due to friction, even though care was taken to reduce the friction. The beams therefore had a vertical displacement close to the wall edge uplift, which is higher than the one at the wall centre. There was also a noticeable increase of strength in this system.

Connection type 4 - large diameter pin:
As expected, no rotations were transferred to the beam and the uplift of the beam followed closely the

wall uplift. There was almost no interaction of the beams with the wall in terms of strength increase.

Connection type 5 - large diameter pin with slotted hole:
This connection was supposed to provide better results than connection type 4, as uplift should be prevented by the slotted hole in the wall. Because of the high friction however, the expected sliding did not occur. In terms of rotation and strength increase the two connections type 4 and type 5 were relatively close and differences can be attributed to construction tolerances.

Connection type 6 - steel comb with slotted holes:
Because of the high friction between the external bolts and the steel element, no sliding occurred on the slotted holes. Due to the length of the steel comb, high rotations were imposed on the beam. This was also reflected in the increase of stiffness of the system compared with no beams.

Connection type 7 - steel-to-steel connection with slotted holes:
This connection yielded the best results as there was no rotation incompatibility. Again the sliding did not occur to the expected degree, but uplift of the beams was slightly less than the uplift of the walls. The system strength is almost unaltered compared with no beams.

Connections which allowed rotation showed only small increase in system strength, additionally the axial forces in the gravity columns were smaller than with other connections. Avoiding rotation incompatibilities has therefore a bigger impact on the system behaviour than vertical displacement incompatibilities. This conclusion could also be drawn in a separate analytical analysis which is still in progress and not included in this paper. Because of the relatively small flexural stiffness of the beams, the vertical displacement incompatibility was often accommodated by beam bending rather than by sliding in the slotted holes. No damage could be observed in any of the connections.

5 CONCLUSIONS

This paper describes the possible displacement incompatibilities between rocking timber walls and timber floor diaphragms and their collector beams. Results of experimental tests provide information on how horizontal forces can be transferred from the diaphragm to the wall through collector beams and how displacement incompatibilities can be reduced or accounted for. Seven different connection details were tested including bolted connections, external blocks, large diameter pins and steel plates with slotted holes, all producing very acceptable behaviour.

No damage was observed in any of the test specimen. For the connection with central bolts, rotation was accommodated by the low rotational stiffness of the fastener layout and the bending of the collector beam. The eccentric bolted connection showed slightly poorer results, this was more because of the eccentric force location than the higher rotational stiffness. The connections with external blocks and the steel comb need improvement because of high friction in the bearing area, which might be less of a problem under dynamic loading. Best results were obtained with the large diameter pin and the steel-to-steel connection with a slot in one of the steel plates, confirming also in the case of timber the earliest proposals developed for precast concrete jointed ductile systems (Priestley et al. 1999; Park et al. 2003). It is worth noting, that this positive performance was mainly due to the fact that no rotation was imposed on the collector beams. The large diameter pins in slotted holes in LVL in fact were not able to slide the expected amount because of high friction and local deformation in the bearing area. For both sliding pin solutions, brass shims (Latham et al. 2013) or teflon pads (PTFE) could potentially reduce friction.

For ordinary design situations an economic and reliable connection is recommended with a group of bolts placed at the centre of the wall. Eccentric connections, if needed because of geometrical constraints or the presence of post-tensioning bars, will induce slightly higher rotations and vertical displacements in the beams. For stiffer floor beams, a large diameter pin or a steel-to-steel connection can provide the best performance. Large diameter pins need to be designed carefully, as embedment strengths for large diameter are not available. The steel-to-steel solution minimizes the interaction between the wall and the beams, but will result in more expensive construction.

Considering the small uplift of the wall for a drift of 2.0% (less than 1/600 of the length of the beam), the vertical displacement incompatibility does not appear to be a major issue in most post-tensioned timber wall systems. In order to reduce imposed rotations, connections should be placed at the centre of the wall, using closely spaced small dowels or one large diameter steel pin. Further testing of a wall with end columns or boundary elements will be carried out in the current research programme. The introduction of dissipative connections between the wall and the beams, as proposed for concrete systems, are also of interest and under consideration for future testing.

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REFERENCES

ACI Innovation Task Group 5 2008. *ITG-5.1-07 Acceptance criteria for special unbonded post-tensioned precast structural walls based on validation testing and commentary: an ACI standard*. Farmington Hills, Michigan, American Concrete Institute.

Brown, A., Lester, J., Pampanin, S., Pietra, D. 2012. Pres-Lam In Practice: A Damage-Limiting Rebuild Project. *Sesoc Conference 2012*. Auckland.

Buchanan, A.H., Palermo, A., Carradine, D., Pampanin, S. 2011. Post-tensioned Timber Frame Buildings. *The Structural Engineer* 89(17): 24-30.

Devereux, C.P., Holden, T.J., Buchanan, A.H., Pampanin, S. 2011. NMIT Arts & Media Building - Damage Mitigation Using Post-tensioned Timber Walls. *Proceedings of the Ninth Pacific Conference on Earthquake Engineering*. Auckland, New Zealand.

Eatherton, M.R. 2010. Large-scale cyclic and hybrid simulation testing and development of a controlled-rocking steel building system with replaceable fuses. *PhD*, University of Illinois at Urbana-Champaign.

Henry, R.S., Ingham, J.M., Sritharan, S. 2012. Wall-to-floor interaction in concrete buildings with rocking wall systems. *New Zealand Society of Earthquake Engineering Conference*. Christchurch.

Latham, D.A., Reay, A.M., Pampanin, S. 2013. Kilmore Street Medical Centre: Application of a post-tensioned steel rocking system. *Steel Innovations Conference*. Christchurch, New Zealand.

Moroder, D., Buchanan, A.H., Pampanin, S. 2013. Preventing seismic damage to floors in post-tensioned timber frame buildings. *New Zealand Society of Earthquake Engineering Conference*. Wellington.

Newcombe, M.P. 2011. Lateral Force Design of Post-Tensioned Timber Frame and Wall Buildings. *PhD*, University of Canterbury.

NZCS 2010. *PRESSS Design Handbook*. Wellington, New Zealand, New Zealand Concrete Society.

Palermo, A., Pampanin, S., Buchanan, A., Newcombe, M. 2005. Seismic Design of Multi-Storey Buildings using Laminated Veneer Lumber. *New Zealand Society for Earthquake Engineering Conference*. Wairakei, New Zealand.

Pampanin, S. 2005. Emerging solutions for high seismic performance of precast/prestressed concrete buildings. *Journal of Advanced Concrete Technology* 3(2): 207-223.

Park, R., Watanabe, F., Fédération internationale du béton. Task, G. 2003. *Seismic design of precast concrete building structures: state-of-art report*. Lausanne, Switzerland, International Federation for Structural Concrete.

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

Sarti, F., Palermo, A., Pampanin, S. 2014. Quasi static cyclic test of 2/3 scale timber single wall and column-wall-column post-tensioned systems. *New Zealand Society of Earthquake Engineering Conference*, Auckland.

Schoettler, M.J., Belleri, A., Zhang, D., Restrepo, J.I., Fleischman, R.B. 2009. Preliminary results of the shake-table testing for the development of a diaphragm seismic design methodology. *PCI Journal* 54(1): 100-124.

Structural Timber Innovation Company (STIC) 2013. *Design Guide Australia and New Zealand - Post-Tensioned Timber Buildings*. Christchurch, New Zealand.

SWG Schraubenwerk Gaisbach GmbH 2012. European Technical Approval ETA-11/0470. Three-dimensional nailing plate (Connector for timberto-steel connections). ETA-Danmark A/S, Charlottenlund, Denmark.