SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

EXPERIMENTAL BEHAVIOUR OF DIAPHRAGMS IN POST-TENSIONED TIMBER FRAME BUILDINGS

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ABSTRACT

Floor diaphragms have an important role in the seismic behaviour of structures, as inertia forces are generated by their masses and then transferred to the lateral load resisting system. Diaphragms also link all other structural elements together and provide general stability to the structure. As with most other structural components, there is concern about damage to floor diaphragms because of displacement incompatibilities. This paper describes two different experiments on engineered timber floors connected to post-tensioned timber frames subjected to horizontal loading.

First a full scale two-bay post-tensioned frame was loaded with lateral loads through a stressed-skin floor diaphragm. Different connection configurations between the floor units on either side of the central column were tested. Secondly a three dimensional, three storey post-tensioned frame building was tested on a shaking table. The diaphragm consisted of solid timber panels connected to the beams with inclined fully threaded screws. For all tested connections, the diaphragm behaviour was fully maintained throughout the testing and no damage was observed.

The test results showed that careful detailing of the floor panel connections near the beam-column-joint and the flexibility of timber elements can avoid floor damage and still guarantee diaphragm action at high level of drifts in post-tensioned timber frame buildings.

INTRODUCTION

This paper describes the experimental behaviour of two different timber diaphragm designs for post-tensioned timber buildings with lateral load resisting systems. The first of these is the experimental test of a two bay post-tensioned frame with a stressed-skin-panel floor under quasi-static loading. The second is a three-storey post-tensioned timber frame building with a solid timber panel floor under dynamic loading.

Multi-storey post-tensioned timber structures are a sustainable and low damage design answer to the growing demand of seismic resistant structures. The Pres-Lam system developed at the University of Canterbury, Christchurch, New Zealand (Palermo et al. 2005; Buchanan et al. 2011) is based on the precast concrete PRESSS technology originally pioneered in the US (Priestley et al.

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1999) and further refined in the last decade at the University of Canterbury (Pampanin 2005; NZCS 2010). The peculiarity of the system is the use of engineered timber products like laminated veneered lumber (LVL) or glued laminated timber (glulam). Post-tensioned frames and walls provide self-centering lateral load resisting systems and special steel elements provide additional dissipation to the structure. These steel elements are the only elements which might need replacement after a major seismic event.

Problem identification

All moment-resisting frame structures are subjected to the effects of beam elongation during cyclic lateral loading. This is independent from the construction material and happens in traditional systems and also in jointed-ductile systems where the beam-column-joint gap opening is desired to provide damping via dissipation devices. The displacement incompatibilities between the floor and the beam-column-joint shown in Figure 1 can cause damage to the floor diaphragm and has the potential to compromise load paths within the structure and hinder seismic resilience.

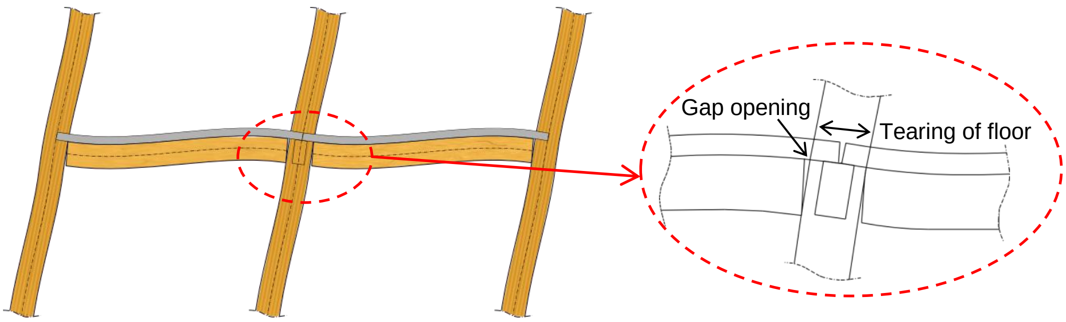


Figure 1. Tearing of the floor due to frame elongation resulting from beam-column-joint gap opening

The mechanism of reinforced concrete frame elongation because of the formation of plastic hinges has been reported since the 1970s (Fenwick and Fong 1979) and further studied in the 1990s (CAE 1999), but implications of these displacement incompatibilities on the design and behaviour of diaphragms have only recently been addressed by researchers (Bull 2004). Experiments by Matthews et al. (2003) simulated the collapse of precast flooring system because of beam elongation and the resulting pushing out of columns and beams (see Figure 2). Subsequent research by Lindsay et al. (2004) and MacPherson et al. (2005) led to detailing improvements to guarantee the diaphragm behaviour in the case of a seismic event; these solutions however still allow substantial damage.

Amaris et al. (2008) proposed two new non-tearing floor solutions. Their design recommendations included complete avoidance of beam elongation by allowing differential movements of the frame with respect to the diaphragm by sliding connection devices or by introducing a top hinge connection at the beam-column-joint. Au (2010), Leslie et al. (2010) and Muir et al. (2012) further developed the latter system and proposed a slotted beam solution, which tends to eliminate frame elongation.

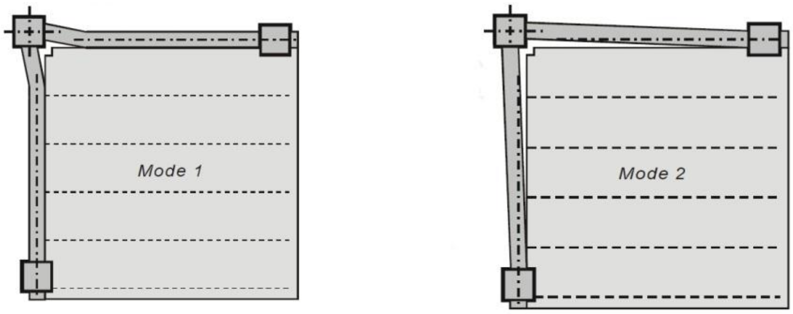


Figure 2. Particular deformation modes because of beam elongation (Matthews et al. 2003)

Pres-Lam structures

Whereas post-tensioned timber frames and walls have been studied in the past (Smith et al. 2007; Newcombe et al. 2010b), little is known about the diaphragm behaviour inside this type of structural system. Initial information regarding the floor diaphragm behaviour is provided by Smith (2008) in the case of a beam-column subassembly connected to a portion of floor slab. Newcombe et al. (2010a) tested a 2/3 scale building under biaxial loading. Both tests used timber-concrete-composite floors, whereas the tests in this paper used a timber-only floor with no concrete topping.

The experimental campaigns presented in this paper examined the integrity of multi-storey structures with timber floor diaphragms when subjected to high levels of interstorey drifts. The flexibility of the timber members and well-designed connections accommodated displacement incompatibilities deriving from beam-column-joint gap openings with negligible damage.

This paper describes the two experimental test setups and discusses the diaphragm behaviour under horizontal quasi-static and dynamic loading. Conceptual design recommendations for timber only and TCC floors are given.

EXPERIMENTAL TEST SETUPS

Two bay post-tensioned frame

The first of the two experimental campaigns discussed is a two bay post-tensioned timber frame built and loaded horizontally at the University of Canterbury. The full scale frame shown in Figure 3 was assembled with an engineered timber-only floor sitting on top of the main beams. The frame was loaded by applying horizontal forces to the floor elements.

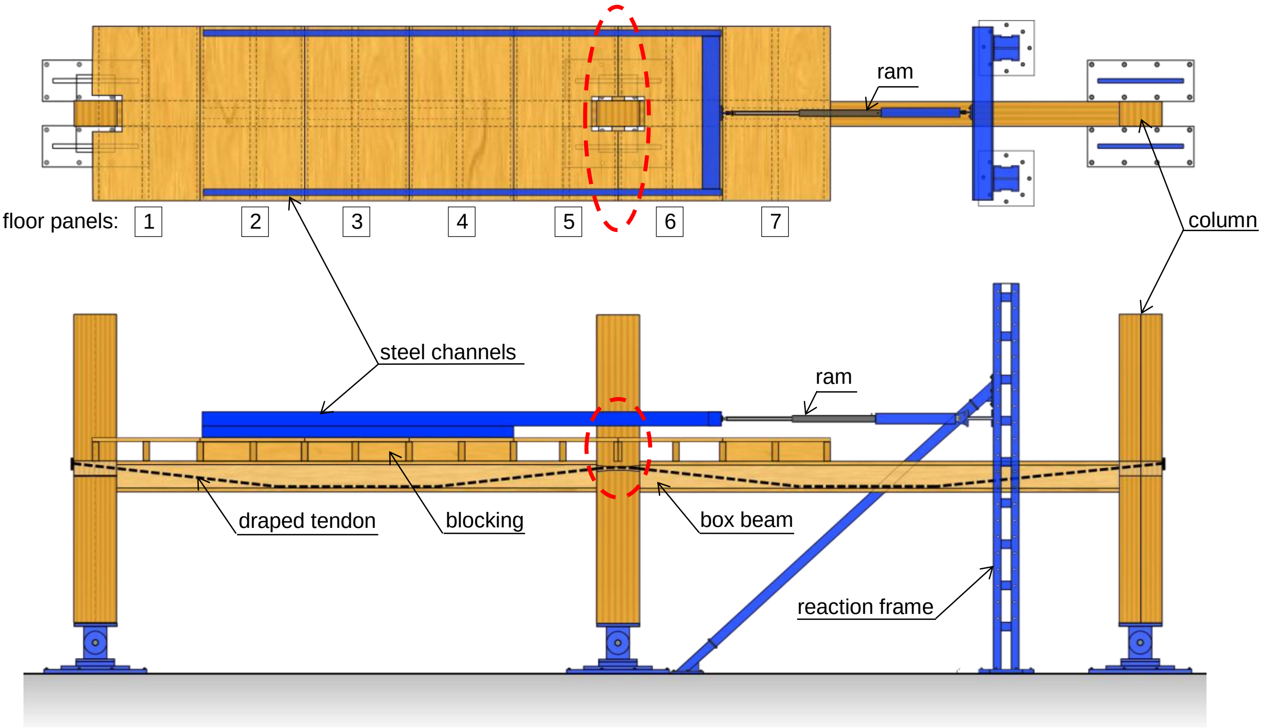


Figure 3. Experimental setup of the two bay post-tensioned LVL frame

The frame with bay lengths of 6 m consisted of 3 solid columns (288 x 500 mm) and two box beams (288 x 360 mm) with webs and flanges made of 45 mm elements. All elements were made of LVL11 (E = 11 GPa). The beams were sitting on steel corbels and connected to the columns by four 7-wire pre-stressing strands (diameter 12.7 mm) tensioned up to 100 kN.

To simulate the timber-only floor diaphragm, seven 2 m long floor panels were mounted on top of the beams. These were designed as stressed-skin T-panels for a span of 7.4 m resisting a dead load of 2 $^{kN}/_{m^2}$ and a live load of 3 $^{kN}/_{m^2}$. The top skin was a 36 mm cross-banded LVL panel, the internal and external joists were 90 x 290 mm and 45 x 290 mm respectively. The joists and, where present, the blocking, were connected to the top skin by nail-gluing, using 3.3 x 90 mm gun-nails at 50 mm centres. The blocking was necessary to transfer the horizontal shear forces from the diaphragm to the beams through panels 2-4 and 7. These 45 x 290 mm blocking elements were connected to the web of the frame beams by steel plates with Ø8/80 mm screws and M10 bolts respectively. The diaphragm was designed for a unit shear force of 20 kN/m; the single floor elements were connected to each other by using 45° inclined Ø6/120 mm fully threaded screws at 150 mm centres.

As the behaviour of the floor at the position of the central column (circled in Figure 3) was of principal interest, the specimen was tested by considering the following setups:

1. Floor elements at the central column were not connected, i.e. left and right portion of floor elements could slide respectively to each other;
2. Panels 5 and 6 were connected at the bottom of the external joists by fully threaded screws (Figure 4 a);
3. All panels were connected at the top of the joist by fully threaded screws (Figure 4 b).

The first setup was necessary to measure the amount of floor gap opening to expect and to obtain benchmark values. The two successive connection details provided the *concentrated* and the *spread gap solutions* respectively. All other panels away from the central beam-column-joint were connected by 45° inclined Ø6/120 mm fully threaded screws at the top of the floor joist to guarantee diaphragm action.

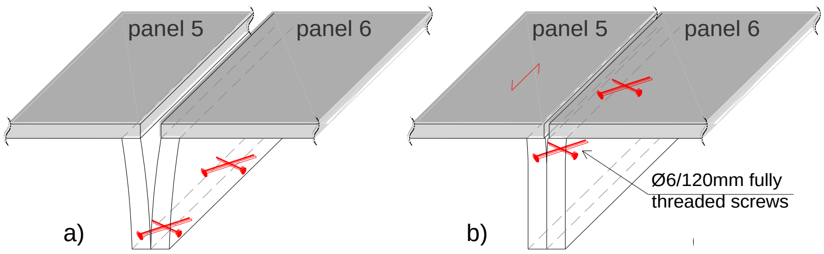


Figure 4. Connection of floor elements at the central beam-column-joint: a) bottom joist connection and b) top joist connection

The frame was loaded horizontally through floor panels 2-4. The quasi-static cyclic loading protocol was based on ACI 374.1-05 (ACI Committee 374 2005), omitting the small cycles in between the three repetitive cycles.

Linear displacement potentiometers were used to measure the gap opening at the beam-column-joints and between the floor panels, as well as any elongation of the panels.

Three storey post-tensioned timber frame building

The second experimental setup was a three-dimensional, three-storey post-tensioned timber frame building made of glulam beams and columns. The specimen as shown in Figure 5 was built at the University of Basilicata, Potenza, Italy in collaboration with the University of Canterbury and was tested under dynamic loading in real time. The timber diaphragms consisted of solid glulam panels connected with vertical screws to each other, and inclined screws to the timber beams on all four sides of the building (as shown in Figure 7).

Figure 5. Experimental setup of the 3 storey post-tensioned glulam frame building; timber flooring, attachment and additional mass arrangement

The prototype structure had a single bay in both directions and was designed with a live load of 3 $^{kN}/_{m^2}$ with the final storey being a rooftop garden. The interstorey height of the building was 3 m and the frame footprint was 6 m by 4.5 m. A scale factor of 2/3 was applied, resulting in an interstorey height of 2 m and a footprint of 4 m by 3 m. The sections of the columns and beams are shown in Figure 6; all elements were made of glulam grade 32h according to EN 1194:1999-05 (Smith et al. 2014). The structure had post-tensioned frames in both directions, but was loaded in the long (4 m) direction.

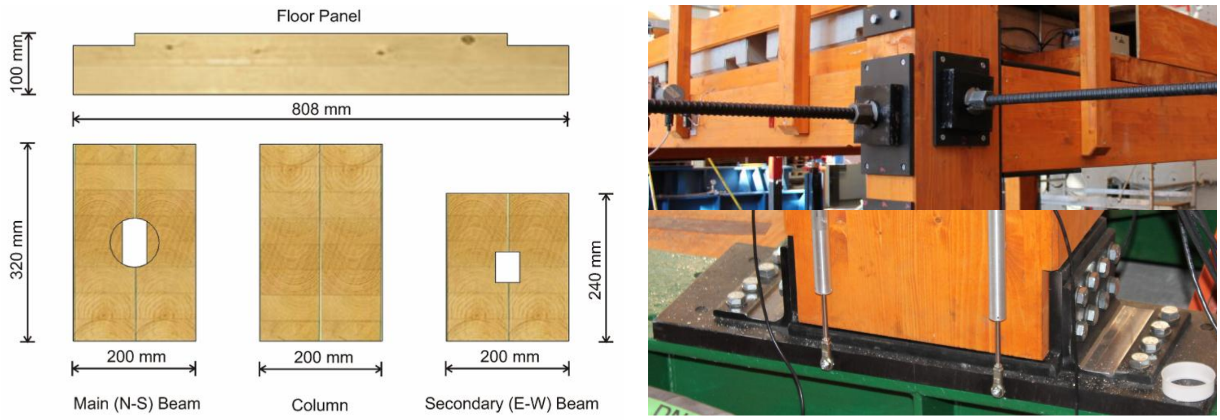


Figure 6. Section sizes used in glulam test frame, beam-column and column-foundation connection details

The timber diaphragms were designed for a unit shear force of 15 $^{kN}/_{m}$ and consisted of 100 mm thick glulam panels connected with a plywood spline placed in a recess and connected with Ø6/80 mm partially threaded screws at 90° every 150 mm as shown in Figure 7. The whole diaphragm panel was connected to the main beams in the loading direction by couples of Ø7/220 mm fully threaded screws at 45° every 200 mm. Perpendicular to the direction of loading the panels were fixed to the secondary beams by Ø6/240 mm partially threaded screws at 45° every 186 mm.

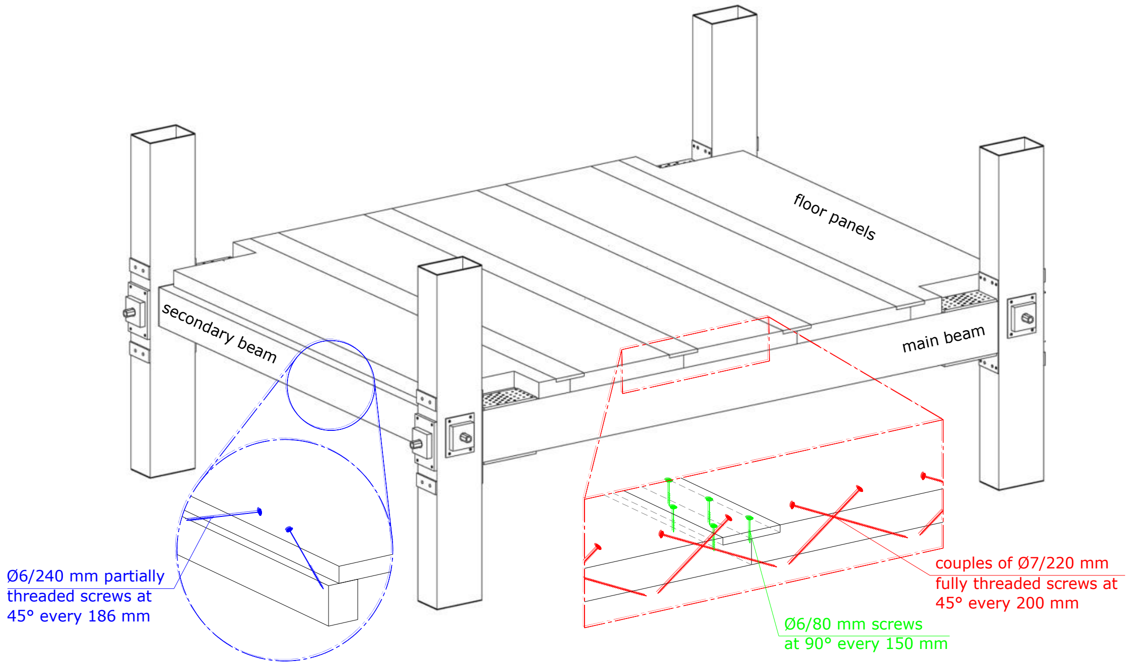


Figure 7. Connection of the panel elements to each other and the diaphragm to the frame

Additional mass was added to the floors to simulate the factored live load and to account for the scaling of the prototype structure. The mass was made up of a combination of concrete blocks and steel hold downs as shown in Figure 5.

The loading input was a set of 7 spectra compatible earthquakes selected from the European strong-motion database (Izmit 1999, Turkey; Montenegro 1979, Serbia; Erzican 1992, Turkey; Tabas 1978, Iran; Campano Lucano 1980, Italy and South Iceland 2000 with two different PGA). The code spectrum was defined in accordance with the current European seismic design code (EN 1998-1:2003 2003) having a ground acceleration of a_g= 0.35 and a soil class B giving a PGA for the design spectrum of 0.44. Because of the 2/3 scale structure, the time of the input was scaled by the same factor, thus altering the input period content by $(2/3)^{0.5}$. The shake table tests were performed with and without additional dissipative steel angle elements with increasing PGA levels.

For further information regarding the design of the specimen including masses, connections and the seismic input refer to Ponzo et al. (2012) and Smith et al. (2013a); more details on the beam-column connection system can be found in Smith et al. (2013b).

RESULTS AND DISCUSSION

Both test specimens underwent a significant quantity of loading cycles without any noticeable degradation and re-centered following testing. Visual inspection proved that no damage occurred to the floors and that the integrity of the diaphragms and their connections was fully maintained.

Two bay post-tensioned frame

Measurements of the diaphragm deformations in the two bay frame showed that the flexibility of the timber elements and the flexibility of the connections between the single panels allowed for the displacement incompatibilities. When the panel elements at the position of the central beam-column-joint were connected at the base of the floor joists, a single concentrated gap formed between the panels as can be seen in Figure 8a. The required displacement is provided by elastic transverse bending of the LVL joists over their height.

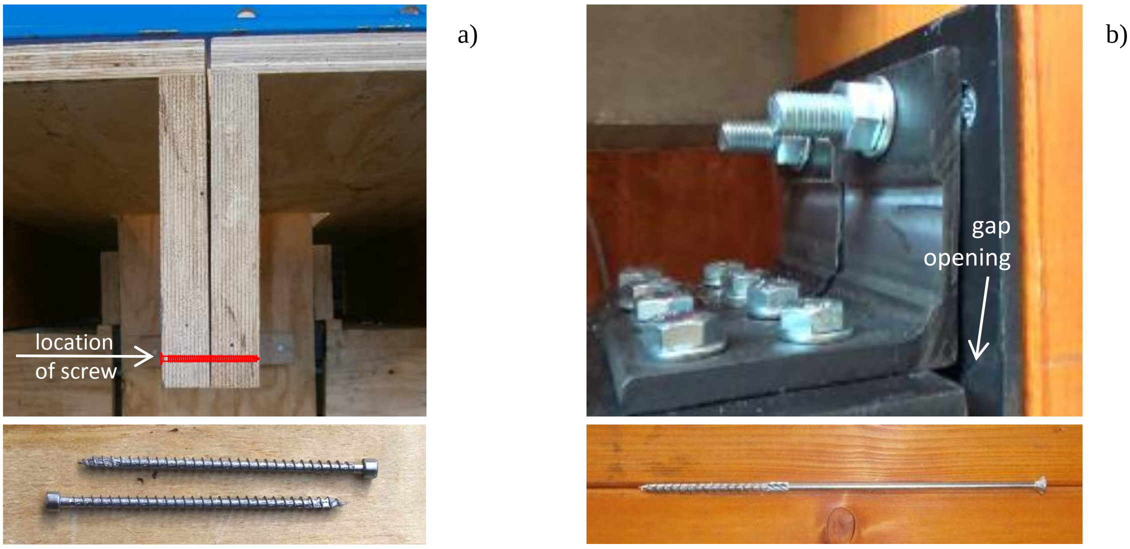


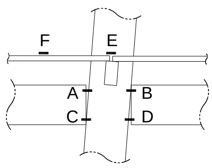
Figure 8. a) Beam-column-joint and floor gap openings with undamaged fasteners of the two-bay frame system and b) beam-column-joint opening with undamaged fastener of the connection to the secondary beam of the multi-storey building

When the floor panels were connected by a much stiffer top joist connection (Figure 4 b), the imposed displacement could not be taken solely by a single concentrated gap, but was spread over several panel joints. As can be seen in Table 1 the magnitude of the panel gap opening in correspondence to the beam-column-joint was smaller than for the previous connection, the remaining required displacement was provided by further gap openings from adjacent panels. Additionally the panels also underwent some elongation.

In both cases, the sum of the panel gap openings and elongations allowed for the gap opening between the beam and the column. Visual inspection (Figure 8a, bottom) showed that screws were not deformed after testing, thus the screws were working in the elastic range. Comparison of the force-deformation response of the bare frame and the frame with the floor panels showed no significant interaction between the frame and the diaphragm. Table 1 summarizes some key values for the three different test setups. For additional test results refer to Moroder et al. (2013).

Table 1: Key values in mm for all three setups for 2.5% drift (values in parenthesis are at 3.5% drift) of the two bay frame

	Beam-column-joint gaps				Panel	
Test Setup	top left **A**	top right **B**	bottom left **C**	bottom right **D**	gap 5-6 **E**	elongation **F**
1	5.72	6.14	7.85	6.13	6.58	0.31
2	5.85 *(9.33)*	5.63 *(8.68)*	7.84 *(11.14)*	6.70 *(10.52)*	3.55 *(6.71)*	1.20 *(1.56)*
3	5.79 *(9.29)*	5.27 *(8.23)*	7.74 *(11.23)*	6.87 *(10.75)*	1.09 *(1.91)*	1.77 *(2.39)*

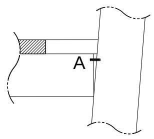


Three storey post-tensioned timber frame building

In the multi-storey structure the imposed displacement incompatibilities led to elastic deformations of the connections of the diaphragms. It was expected that most of the displacement would be provided by the connection of the panels to the transverse beam, as the screws are relatively flexible in shear. Less displacement was expected to happen at the connections between the single floor panels, as the panels were connected to the main beam and would follow its movement. Additional instrumentation is planned for future testing in order to register directly these movements. Several fasteners were extracted from the specimen after testing and no damage to the fasteners was observed as can be seen at the bottom of Figure 8b which shows the Ø6/240 mm partially threaded screws from the connection to the secondary beam. Also the Ø6/80 mm screws in between the panels and the Ø7/220 mm fully threaded screws from the connection to the frame beam were perfectly straight after testing. Table 2 provides the maximum values of the first storey drift and the maximum beam-column-joint gap openings.

Table 2: Maximum drift and gap opening values at the first storey beam-column-joint for the three storey structure under dynamic loading

	1st floor drift [%]	**A** Beam-column gap opening [mm]
Without dissipation at 75% PGA intensity	2.77	4.6
With dissipation at 100% PGA intensity	3.50	4.3



The rocking of the frames also caused some vertical displacement incompatibility of the diaphragms. As shown in the sketches in Figure 9 the transverse beam rotated with the columns and as the diaphragm panel was fixed to it, it was forced to follow this movement. As a result, the diaphragm panel was pushed up and down relatively to the frame beams. During testing the floor was pushed up when the structure underwent negative drift (Figure 9 middle) and was pushed down when undergoing positive drift (Figure 9 right). It is assumed that latter was caused by the relatively stiff connection of the screws working axially and the relatively low compressive stiffness of timber perpendicular to grain. Future testing with additional instrumentation will capture this movement in a more detailed manner. Because of the relatively small movement, the flexibility of the connections and the possibility of the panel to rotate out-of-plane with respect to the adjacent panel, this relative movement was easily accommodated for. For additional test results refer to (Smith et al. 2014).

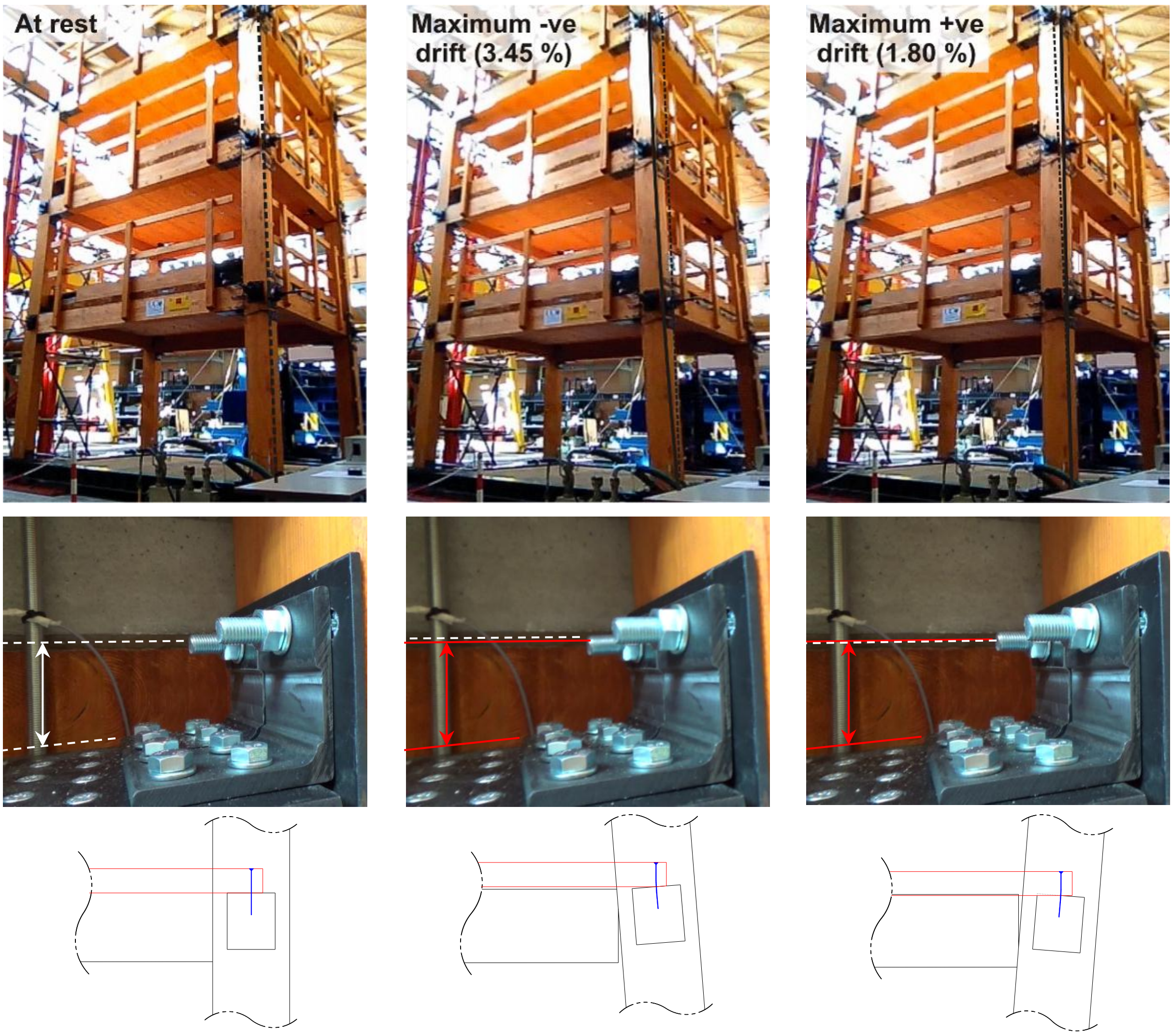


Figure 9. Photos and sketches showing maximum positive and negative drift response of the structure and respective uplifting of the floor diaphragms

DESIGN RECOMMENDATIONS

Independently from the type of timber flooring chosen (stressed-skin T-panels or solid timber panels), the displacement incompatibilities were accommodated for by the flexibility of the connections and to a lesser extent by the flexibility of the timber elements. Therefore it is important to design a connection which allows for the required movement throughout a seismic event without compromising its capacity also for future events.

Stressed-skin T-panels

For a floor setup with stressed-skin panels, the suggested panel connection depends on the flexibility of floor finishings and linings of adjacent internal and external walls to move with the floor (see Figure 10):

- *Concentrated floor gap:* The required deformation occurs mainly in a single gap between floor panels, which requires attention. If the floor joists are flexible enough in transverse bending, a bottom flange connection with screws is sufficient. The connection still needs to guarantee the full shear transfer between the panels. If required, special steel elements, which allow the panels to move apart while still transferring the shear forces, can be used. Seismic gaps in the floor finishing and the wall linings may have to be provided.
- *Spread floor gaps and panel elongation*: All floor panels can be connected to each other by metallic connectors like nails or small diameter screws, which give some local flexibility. The connections need to guarantee the full shear transfer between panels. Gluing to connect floor panels should not be used, as it results in a very stiff and brittle connection, which cannot accommodate required deformations. The panels close to the disturbed area should not be directly connected to the beam, as this would prevent the development of gap openings and panel elongations further away from the beam-column-joint. The floor finishing should be chosen to be elastic enough to allow for the formation of spread gaps or might require some cosmetic repair after a major seismic event.

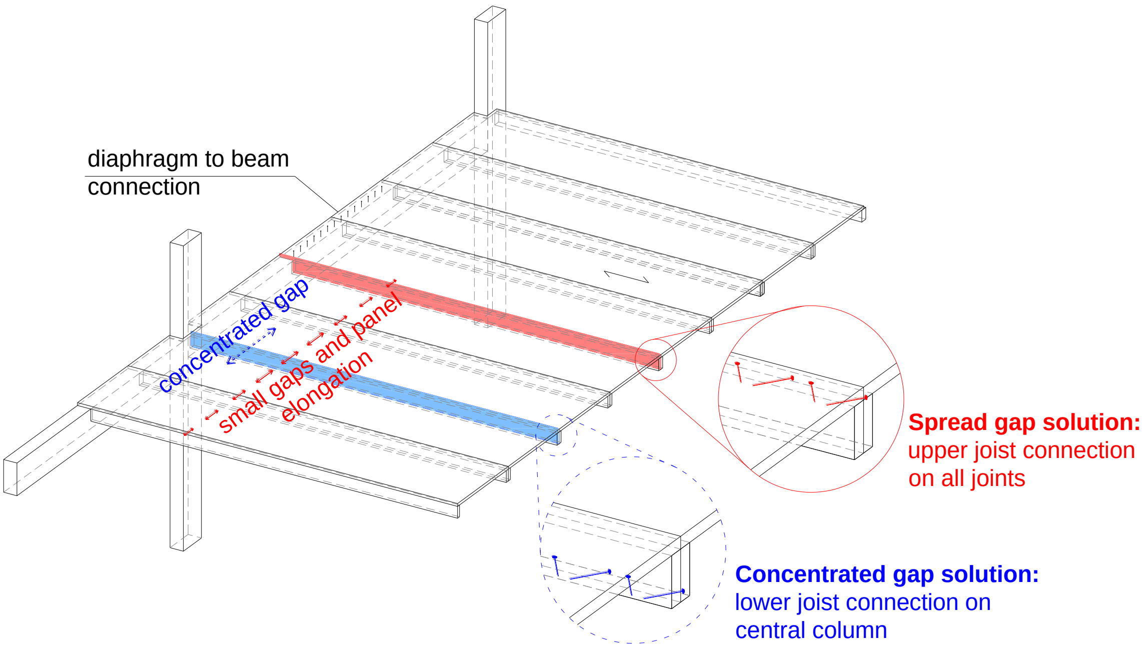


Figure 10. Alternative designs for a concentrated floor gap solution (blue - dashed) and a spread floor gap and panel elongation solution (red - solid) in an stressed-skin timber floor.

Solid timber panels

For a floor setup with solid timber panels running perpendicular to the frame direction, the displacement demand should be provided mainly by the connection of the panels to the transverse beam. This can be achieved by the use of a connection with inclined screws as shown in Figure 11. In the case of gap opening the screws will deform elastically in dowel action but will keep transferring shear when the seismic action runs perpendicular to the frame direction. Only nominal panel elongation can be expected, however the connections between the single panels may contribute to a certain extent. This solution is therefore conceptually the same as the concentrated floor gap solution mentioned above, where the panel joint should be conveniently located at the transverse beam in case of multiple frame bays. The seismic shear forces should be transferred to the longitudinal beams with inclined fully threaded screws as shown in Figure 11.
To achieve some panel gap spreading on panels further away from the beam-column-joint location, the panels close to the disturbed area should not be connected to the main beams. The remaining panels however will need to transfer bigger diaphragm shear forces to the main beam.

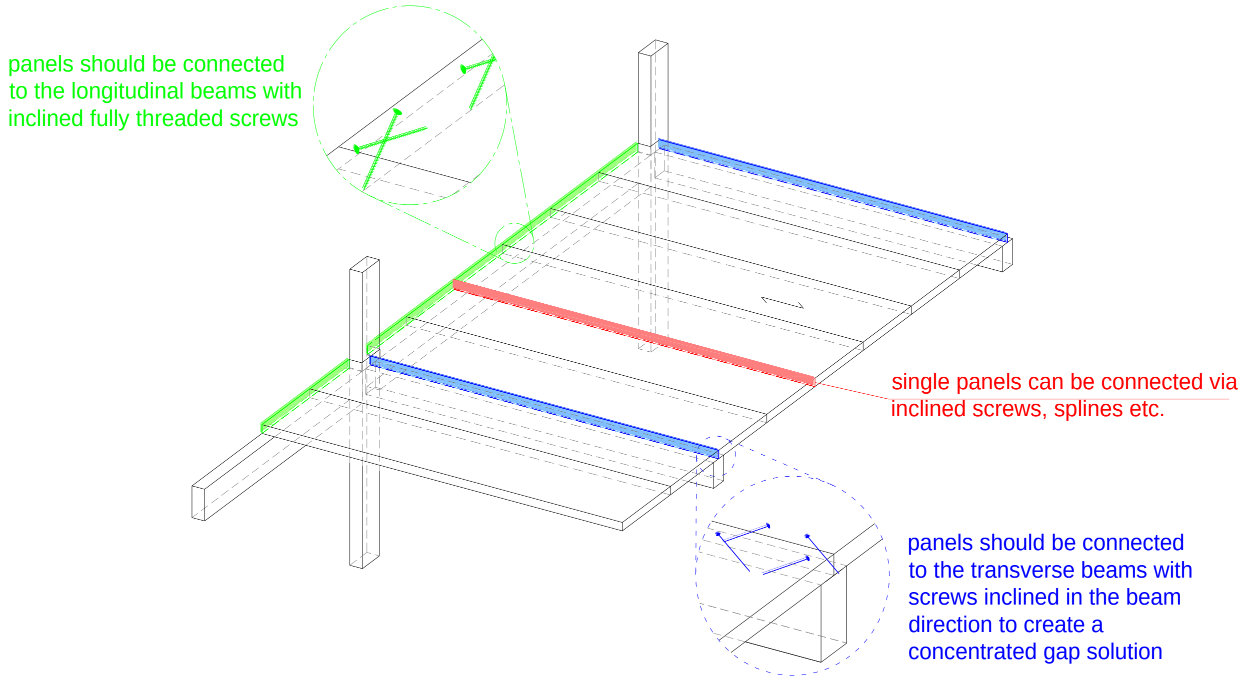


Figure 11. Design for solid panels

Timber-concrete-composite floor

Timber-concrete-composite floors have not been tested in this testing programme, but it is suggested to design them similar to the concentrated floor gap option. The concrete topping should be pre-cracked along the required gap line. Instead of providing continuous steel reinforcement over the crack, the continuity of the diaphragm can be obtained by unbonded rebars. In the case of gap opening, the unbonded part of the rebars will elongate elastically providing the required displacement demand. Shear can be transferred via dowel action if the seismic action occurs perpendicular to the frame direction. Contrary to the timber-only solution, the deformation of the steel rebars may increase the strength and stiffness of the frame.

CONCLUSIONS

This paper described: the behaviour of timber only diaphragms in post-tensioned multi-storey timber frame buildings, the experimental setup and results of a two bay frame tested at the University of Canterbury under quasi-static cyclic loading and the design and testing of a three-dimensional, three-storey, post-tensioned frame building dynamically tested at the University of Basilicata. Based on experimental testing, conceptual design recommendations for timber floor diaphragms have been suggested.

Both test specimens underwent numerous loading cycles without noticeable damage and displacement measurements as well as visual inspection of the diaphragms and their connections suggest that:

- the displacements created by the beam-column gap openings were accommodated by the diaphragm due to the flexibility in the connections and, to a lesser extent, by the elongation of the timber members;
- the beam-column gap openings were not restrained by the presence of the diaphragms.

The following different connection details are proposed:

- *Concentrated floor gap* at the position of the beam-column-joint, in the case of stressed-skin panels with rigid floor finish, solid timber panels and timber-concrete-composite floor (latter however was not been tested in this test programme);
- *Spread floor gaps and elongation* of the floor diaphragm over several floor panels in the case of a flexible floor finish.

ACKNOWLEDGEMENTS

Acknowledgments are given to Dr. Wouter van Beerschoten and Dr. Alessandro Palermo for providing information and design of the post-tensioned frame. The authors also acknowledge the financial support from the Structural Timber Innovation Company (STIC) and FederlegnoArredo (FLA).

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