UNIVERSITY OF CANTERBURY
Te Whare Wānanga o Waitaha
CHRISTCHURCH NEW ZEALAND

LONG-TERM PERFORMANCE OF POST-TENSIONED LVL FRAMES AND WALLS

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Authors:

David Yeoh
David Carradine
Alessandro Palermo
Hugh Morris (University of Auckland)
Rijun Shrestha (University of Technology Sydney)

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Background

This report contains a collective summary of six long-term experiments being performed on post-tensioned LVL frames and walls at the University of Canterbury, University of Auckland and University Technology of Sydney from 2007 to 2012. Although some tests reported herein were before the formation of STIC (Structural Timber Innovation Company Ltd.) or did not come under the umbrella of STIC, most of the tests were funded by STIC. The purpose of this report is to provide an overview of the aforementioned long-term tests, report on important summarised findings and lessons, and provide current progress status. Where further details are required, the respective references should be sought.

Introduction

The loss of pre-stressing force in frames and walls subjected to unconditioned environment is affected by several phenomena:

(1) Relaxation of steel tendons;
(2) "Pure" creep of timber parallel to grain (in beams and walls);
(3) "Mechano-sorptive" creep of timber parallel to grain (in beams and walls);
(4) "Pure" creep of timber perpendicular to grain (in columns);
(5) "Mechano-sorptive" creep of timber perpendicular to grain (in columns);
(6) Shrinkage/swelling of timber parallel (in the beams) and perpendicular to grain (in the columns) because of moisture and thermal variations; and
(7) Thermal expansion/contraction of steel tendons.

"Pure" creep is deformation caused by load duration. It is primarily dependent on loading time.

"Mechano-sorptive" creep is deformation caused by the interaction of loading and changes of moisture content in timber. It is not directly dependant on loading time.

(Davies, 2007; Toratti, 1992; Hanhijärvi and Hunt, 1998; Ranta-Maunus, 1975)

Experiment 1 – Long-term behaviour of pre-stressed LVL members (2007-2008)

a. Background

This experiment was carried out by Matthew Davies as part of his Bachelors Degree in Civil and Natural Resources Engineering third year project at the University of Canterbury to address issues regarding the viability of multi-storey pre-stressed timber structures, specifically the reduction in pre-stress load over time (Davies, 2007; Davies and Fragiacomo, 2008; Davies and Fragiacomo, 2011; Fragiacomo and Davies, 2011).

Two sets of tests ran in parallel, with one set subject to an uncontrolled, heated, indoor environment and the other subject to heated, controlled, indoor environmental conditions (constant RH 50 % and temperature 20 °C). The test programme included full-scale specimens, reduced scale specimens and small blocks of laminated veneer lumber (LVL) (Figures 1.1 and 1.2). The specimens were loaded for one year. All relevant parameters (strain, deflection, tendon load, timber moisture content, environmental temperature and relative humidity) were monitored throughout the time under load. In order to measure the recovery, monitoring continued for a further three months following unloading of specimens.

Test specimens had a box-shaped cross section and the stressing was applied through the centre of the section. Large scale specimens were loaded in bending with, and without applied pre-stress. Other large specimens and some smaller specimens were assembled to replicate the layout of a two-bay, moment-resisting frame. These specimens were pre-stressed with the tendon running through the beams and continuing through the columns; thus, loading the LVL both perpendicular and parallel to the grain. Creep tests were performed on small blocks of LVL subject to compression parallel or perpendicular to the grain.

Figure 1.1 Post-tensioned LVL beams tested under bending (left) and post-tensioned LVL frame specimens (right) in an uncontrolled heated indoor environment.

Figure 1.2 Post-tensioned frame specimens in a controlled environment (left) and small LVL specimens loaded under compression (right).

b. Objective

The objectives of this experiment were to investigate the creep properties of LVL parallel and perpendicular to grain and potential losses in pre-stress force of post-tensioned LVL frames.

c. Pre-stress force and stresses

Initial pre-stress force = 106 kN (corresponds to 70 % strength in a seven-wire 12.7 mm tendon permitted by NZS 3101:2006)

Area in each beam = 38,925 mm^2

Average stress in each beam = 2.72 N/mm^2 (Corresponds to 6 % of compressive strength parallel to the grain of LVL)

d. Important findings of experiment

1. The creep deformation in compression perpendicular to grain was about 4 times greater than the creep deformation parallel to grain.
2. A marked pre-stressing force reduction from 106 kN to 102 kN occurred during the first 80 days at constant relative humidity, particularly in specimens with columns.
3. After 1 year, in a controlled environment there was a 1.4 % loss of pre-stress in LVL beam loaded parallel to grain, and 7 % pre-stress loss in the frame which had 11% of its length loaded perpendicular to grain due to the presence of columns.
4. There was a larger pre-stress loss after 1 year in an uncontrolled environment. Losses were 2 % in the beam and 9 % in the frame.
5. Extrapolating to the end of the service life (50 years), for the LVL beam loaded parallel to grain an 8 % loss was predicted, and a 34 % loss in the frame.
6. This compares to predicted losses of 1 % after 1 year and 3.5 % after 50 years, for steel strand on its own.
7. There was a ± 2 kN (approximately 1.8 %) seasonal fluctuation due to shrinkage and swelling of LVL in the uncontrolled environment. Shrinkage due to moisture reduction

(desorption) resulted in pre-stress loss and swelling due to moisture increase (absorption) resulted in recovery of the pre-stressing force.

8. Pre-stress losses are caused by both pure creep and mechano-sorptive creep of the LVL parallel and perpendicular to the grain; relaxation of the pre-stressing strand; and environmental variations. Of these, the proportion of timber loaded perpendicular to grain relative to the timber loaded parallel to grain has the greatest influence on pre-stress losses.
9. An analytical solution extended from Eurocode 2 formulae for reinforced concrete members pre-stressed with unbonded tendons based on the age-adjusted effective modulus method was developed and compared with experimental values. The equation is a useful design tool for estimating the long term deformation and pre-stress losses of LVL members.

e. <u>Progress to date</u>

Experiment was completed in 2008.

f. <u>Important lessons</u>

1. The amount of pre-stress losses in a frame is significantly affected by the ratio between the length of LVL loaded perpendicular to the grain and the total length of the frame.
2. In an uncontrolled, heated, indoor environment, the pre-stress loss in an LVL beam loaded parallel to grain was 2 % after 1 year and predicted as 8 % after 50 years. This is much less than a frame where 11 % of its length was loaded perpendicular to grain due to the presence of columns, giving pre-stress loss of 9 % after 1 year and 38 % after 50 years. The losses predicted after 50 years are indicative and possibly conservative as they are only based on one year test results.
3. The length of LVL loaded perpendicular to grain should be limited as much as possible.
4. The amount of pre-stressing loss is significantly affected by the ratio between the axial stiffness of the timber frame and the axial stiffness of the steel tendon. The smaller the ratio, the larger the losses, and vice versa.
5. Hence, it is recommended to reinforce the perpendicular to the grain LVL (column joint) with screws, steel inserts, armouring or cross-banded LVL.

Experiment 2 – Long-term monitoring of pre-stressed LVL beams (2010 – continuing)

a. <u>Background</u>

This experiment was carried out in a shed belonging to Robert Finch in West Melton, 20 km west of Christchurch (Giorgini, 2010; Giorgini et. al., 2011).

The experimental setup was initially carried out by Simona Giorgini beginning April 2010 which comprised of two 9 m long simply supported straight tendon LVL box beams (Beams 1) and two half-scale 9 m long beams having a support in the centre, thus creating two 4.5 span beams that were continuous over the centre support (Beams 2). These half-scale specimens utilised a draped tendon profile and required uplift restraint at the centre support. While all four beams were stressed parallel to the longitudinal axis, only one beam of each set was loaded with concrete blocks which simulated the gravity load at serviceability limit state for a heavy floor load such as timber-concrete composite floor in the larger beam and a light floor load such as hollow core timber floor in the smaller beam.

The progress of this long term test was interrupted by the 2010 and 2011 Christchurch earthquakes and continued aftershocks. The beams also encountered crushing parallel to the grain failure due to LVL crushing at the anchorages, and this failure was more severe for the ends of the beams closer to the partially open end of the building in which they were located (Figure 2.1). Such failure was likely due to the severe environmental conditions in the shed corresponding to Service Class 3, which was not considered or expected in the initial design of the end blocks. This failure was remedied by cutting off the ends of each beam and gluing and screwing in LVL blocks to more effectively distribute loads from the steel anchorage plates onto the ends of the beams and to avoid the effects of eccentric loading of the plates. Approximately 400 mm were cut off from the large beams and 250 mm from the small beams following removal of loading blocks and de-stressing and removal of the steel tendons. In each beam an LVL block was glued in using resorcinol adhesive mixed on-site and screws were installed at 45° to provide additional strength and also clamping pressure for the glue. Once the adhesive was cured the beam ends were flattened and the beams were re-stressed and then loaded (Figure 2.2). A chronology of the events that happened throughout this experiment is summarised in Table 2.1.

Table 2.1 Summary chronology of experiment

Date	**Events**
April 2010	Beams 1 and 2 were set up, stressed and loaded with concrete blocks. Monitoring begun.
September 2010	Earthquake happened. Beams 1 fell over and Beams 2 shifted on supports. Following this, all beams including Beams 2 were unloaded and de-stressed. Monitoring halted.
November 2010	Re-setup of all beams, stressed and loaded again. Resumed monitoring.
February 2011	Earthquake happened. However, no damage was incurred and monitoring continued.
May 2011	Discovered failure at anchorage block in the loaded Beam 1 and loaded Beam 2. Subsequently, all beams were unloaded and de-stressed again. Monitoring stopped.
June - Dec 2011	Remedial efforts to repair the beams.

	The ends of the beams were cut off. A mid height LVL member were glued at each end of all the beams and reinforced with 3 to 4 screws (SFS screws in Beams 1 and Type 17 screws in Beams 2) installed at 45° to the longitudinal axis so the beams on each side. These additional members were positioned such that they lined up with the top edge of the steel plate in order to increase the bearing area.
January 2012	Completed repair on Beams 1. Set up again, stressed and loaded. Monitoring resumed.
Mar - Apr 2012	Completed repair on Beams 2. Set up again, stressed and loaded. Monitoring resumed.
Apr 2012 - present	Monitoring continues

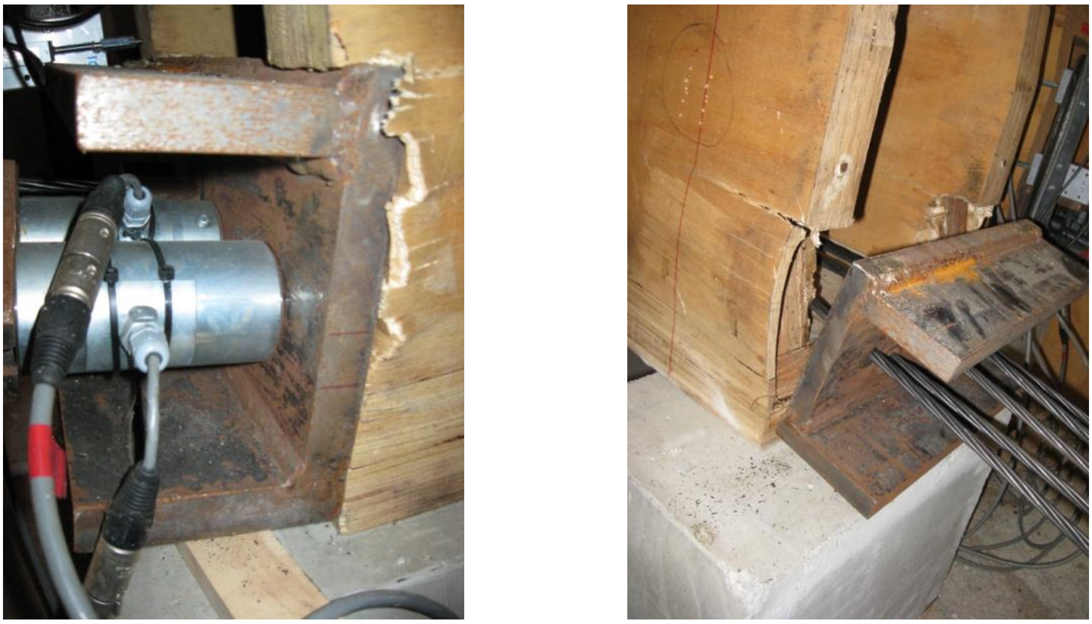


Figure 2.1 Crushing failure due to eccentric loading and insufficient bearing area between the steel anchorage block and LVL.

Fig 2.2 Setting up of Beams 2 (left) and Beams 1 after repair of the anchorage block (right).

b. Objective

The objective of this experiment was to investigate the behaviour of longitudinally post-tensioned LVL beams over the long-term considering the effects of post-tension losses and static loads.

c. Pre-stress Force and Stresses

Beam 1 (Six 12.7 mm diameter 7 wire strand tendons per beam):
Pre-stress force = 780 kN (130 kN for each tendon)
Effective area = 49,230 mm^2
Average stress = 15.8 N/mm^2
(Corresponds to 35 % of of LVL compressive parallel to grain strength)
Local stress at failure (only webs) = 59 N/mm^2

After repair with additional flange member and screws,
Effective area = 66,492 mm^2
Average stress = 11.7 N/mm^2
(Corresponds to 26 % of of LVL compressive parallel to grain strength)

Beam 2 (Three 12.7 mm diameter 7 wire strand tendons per beam):
Pre-stress force = 390 kN (130 kN for each tendon)
Effective area = 22,500 mm^2
Average stress = 17.3 N/mm^2
(Corresponds to 39 % of of LVL compressive parallel to grain strength)

After repair with additional flange member and screws,
Effective area = 27450 mm^2
Average stress = 14.2 N/mm^2
(Corresponds to 31.5 % of of LVL compressive parallel to grain strength)

d. Important findings of experiment

1. Useful experimental data were lost due to the 2010-2011 earthquakes and continuous aftershocks.
2. It was shown that it is critical for the design of post-tensioned members to provide adequate bearing for the anchorage plates along all four sides of the plate and to also consider the worst possible scenario when selecting a service class.
3. Analytical design procedures to predict long-term post-tensioning loss have been presented (Giorgini, 2010; Giorgini et. al., 2011; Davies and Fragiacomo, 2011; Fragiacomo and Davies, 2011).

e. Progress to date

This experiment is on-going. All results collected to date have yet to be analysed.

f. Important Lessons

1. Provide extra bearing for anchorage plates and consider worst possible case in selecting service class.

Experiment 3 – Long-term performance of post-tensioned LVL buildings (2009)

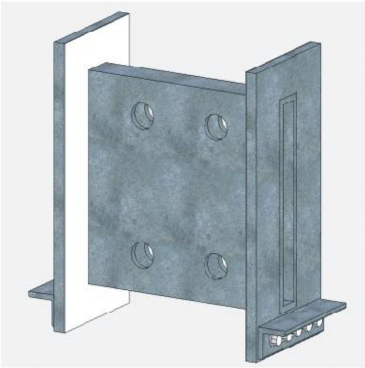
a. Background

This experiment was carried out by Ashley Neale as part as part of his Bachelors Degree in Civil and Natural Resources Engineering third year project at the University of Canterbury (Carter, 2010; Neale, 2009; Newcombe, 2011).

The test specimen was a 2/3rd scale 2 storey post-tensioned laminated veneer lumber (LVL) building built in the laboratory of University of Canterbury and subjected to an extensive quasi-static seismic testing programme (Figure 3.1). The building comprised of post-tensioned moment resisting frames in one direction and post-tensioned shear walls in the other direction, and the floors were timber-concrete composite. Concerns about crushing in the column due to compression perpendicular to the grain at the beam-column interface and the post-tensioning losses in the tendons were the motivations behind this experiment.

Figure 3.1 A 2/3rd scale 2 storey post-tensioned laminated veneer lumber (LVL) building built in the Structures Laboratory of the University of Canterbury

The beam-column connections at Level 1 were fully armoured with steel plates and a hollow steel tubes through the columns (Figure 3.2). At Level 2, the beam-column connections were reinforced diagonally with SPAX screws (Figure 3.3). Forces in the tendons, creep strain effects in the frames both parallel and perpendicular to the grain primarily at the beam-column interface were recorded for a period of 75 days. The average measured temperature was 14 °C, relative humidity 50 % and LVL moisture content 14 %, but these were not controlled during the experiment.



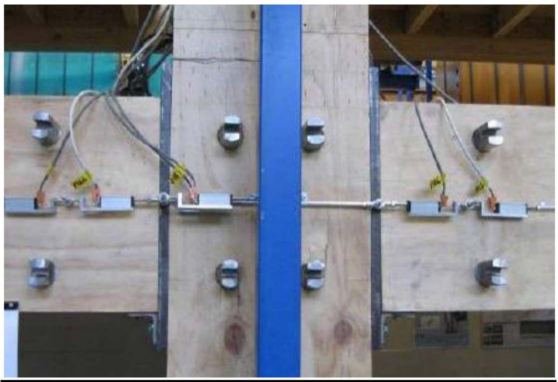
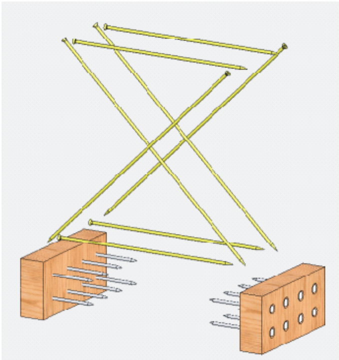


Figure 3.2 Fully armoured beam-column joint on Level 1



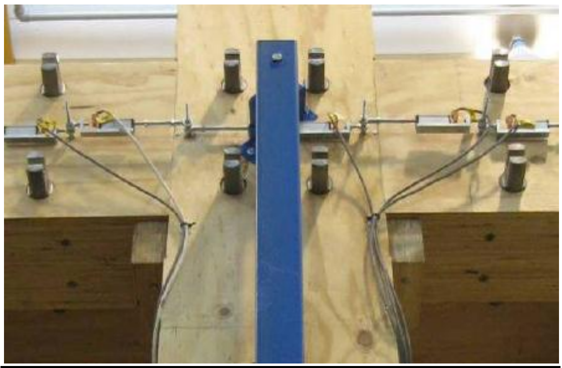


Figure 3.3 SPAX screw reinforced beam-column joint on Level 2

b. Objective

The objective of this experiment was to investigate the long-term performance of a post-tensioned LVL building related to the possible losses in tendon forces due to the effects of compression parallel and perpendicular to the grain.

c. Pre-stress Force and Stresses

In the wall (Five 12.7 mm diameter 7-wire strand tendons):
Pre-stress force = 465 kN (93 kN for each tendon)
Wall cross-sectional size = 800 × 144 mm
Assume that steel plate covers the full cross-section of member
Effective area = 91,200 mm^2

Average stress in wall = 5.10 N/mm^2
(Corresponds to 11 % of LVL compressive parallel to grain strength)

In the frame (Four 12.7 mm diameter 7-wire strand tendons):
Pre-stress force = 372 kN (93 kN for each tendon)
Beam and column cross-sectional size = 400 × 244 mm
Assume that steel plate covers the full cross-section of member
Effective area in beam and column = 84,800 mm^2

Average stress in beam = 4.40 N/mm^2
(Corresponds to 10 % of LVL compressive parallel to grain strength)
Average stress in column = 4.40 N/mm^2

(Corresponds to 37 % of LVL compressive perpendicular to grain strength)

d. Important findings of experiment

1. In the fully armoured connection, only 1 kN of loss in the tendon force was recorded corresponding to a loss of 1 %. All of this was due to shortening of the beams parallel to the grain.
2. In the screw reinforced connection, there was a loss of approximately 8 kN in the tendon force corresponding to 10 % of loss in force over a period of 75 days.
3. The creep strain parallel to the grain measured along the beams of the frames was reported to be extremely minimal (in the range of -0.002 % to 0.006 %). The creep strain on Level 2 was found to be 1.5 times larger than that of Level 1 possibly due to different temperature change and exposure to direct sunlight.
4. The creep strain perpendicular to the grain measured across the fully armoured column at Level 1 measured between -0.01 % to 0.015 % compared to 0.25 % in the screw reinforced column at Level 2.
5. The creep strain at the beam-column interface of the fully armoured connection was found to be 2.5 times larger than the parallel to the grain creep strain. This could possibly be due to buckling in the steel inserts within the armoured connection.
6. The creep strain at the beam-column interface of the screw reinforced connection was significantly larger than the creep strain of the fully armoured connection.

e. Progress to date

This experiment was completed in 2009.

f. Important Lessons

1. Over 75 days, the frame with fully armoured connection showed only 1 % of loss in the tendon force compared to 10 % of loss in the frame with screw reinforced connection.
2. The use of fully armoured connection in beam-column joints was found to be more effective than screw reinforcement in overcoming the effects of crushing perpendicular to the grain the column and losses in the tendon force.
3. The steel plate at the beam-column interface should cover the full width of the column to prevent stress concentration effects on the column face.

Experiment 4 – Long-term monitoring of post-tensioned frames and walls in the EXPAN building (2012)

a. Background

In August 2010, following the completion of an extensive quasi-static testing programme on the pre-EXPAN 2/3rd scale 2 storey post-tensioned laminated veneer lumber (LVL) test building in the laboratory (same test building explained in Experiment 3), the building was deconstructed and reconstructed on the University of Canterbury campus, now used as the STIC office (Holmes Consulting, 2011; Smith, 2011).

Although it had been planned, no instrumentation was installed to monitor potential shortening and post-tensioning losses in the frames and walls. However, to date, readings of total lengths of the frames and walls are being made using a laser range finder. Based on these readings, the post-tensioning losses in the frames and walls for the first year have been estimated (Figure 4.1).

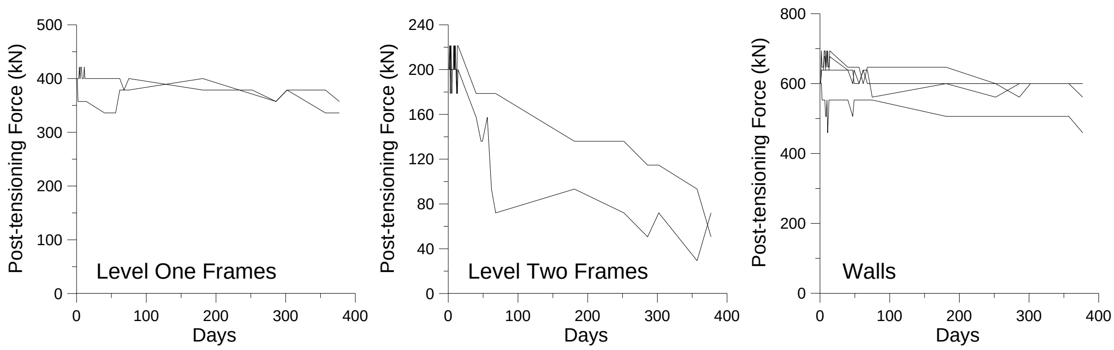


Figure 4.1 Post-tensioning force in frames and walls for the first year in EXPAN Building (Smith, 2011)

A 10% loss was estimated for walls where LVL is loaded parallel to the grain and for the frames in Level 1 where the beam-column joints were fully armoured with steel inserts to prevent crushing in the column perpendicular to the grain. The Level 2 beams were lifted up to the top of the columns in the new building, to a level where the beam-column joints were completely unreinforced. Major losses up to 50 % have been reported at this level. Note that these estimations were based on crude measurements and may contain significant errors. Nevertheless, they are indicative of the trend and extent of loss in non-armoured columns that are loaded in perpendicular to grain compression.

During the construction of the building, the LVL shear wall with a cross-section of 150(*w*) × 400(*l*) mm was post-tensioned to a design force of 300 kN per bar. The post-tensioning operation had to be stopped due to severe crushing of the LVL at the top of the wall. Bearing plates were sized 130(*w*) × 195(*l*) × 25(*t*) mm, which did not cover the full section of the wall. This caused instant crushing as loads were applied. Subsequently, a plate of size 150(*w*) × 400(*l*) × 25(*t*) mm which covered the full wall section was used (Figure 4.2) for all four walls.

At this date (August 2012), efforts are under way to instrument the walls, frames and floors in the building to monitor future long-term deformations and losses in post-tensioning elements (Figure 4.3). To date, some crushing due to compression perpendicular to the grain in the columns at the Level 2 unreinforced beam-column joints were found (Figure 4.4).

Figure 4.2 LVL shear wall with small (top) and large (bottom) bearing plate

Figure 4.3 Long-term instrumentation of the EXPAN Building for frame (left) and floor (right)

Figure 4.4 Perpendicular to the grain crushing at the bottom of the beam-column interface located at Level 2 Column 2 (left) and Level 2 Column 6 (right)

b. Objective

The objective of this experiment is to monitor the long-term performance of the EXPAN building. The focus will be on deformation both parallel and perpendicular to the grain, and post-tensioning losses in walls and frames.

c. Pre-stress Force and Stresses

In each wall (Two 1030 Macalloy bars 26.5 mm diameter):
Pre-stress force = 600 kN (300 kN for each bar)
Wall cross-section size = 800 × 144 mm
Steel plate covers the full cross-section of member
Effective area = 91,200 mm^2

Average stress in wall = 6.60 N/mm^2
(Corresponds to 15 % of LVL compressive parallel to grain strength)

In the frame at Level 1 armoured connection (Two 1030 Macalloy bars 26.5 mm diameter):
Pre-stress force = 400 kN (200 kN for each bar)
Beam and column cross-section size = 400(d) × 244(b) mm
Steel plate covers the full cross-section of member. Plate size = 500(d) × 244(b) × 20(t) mm
Effective area in beam = 84,800 mm^2
Effective area in column = 109,200 mm^2

Average stress in beam = 4.72 N/mm^2
(Corresponds to 10.5 % of LVL compressive parallel to grain strength)
Average stress in column = 3.66 N/mm^2
(Corresponds to 31 % of LVL compressive perpendicular to grain strength)

In the frame at Level 2 unreinforced connection (Two 1030 Macalloy bars 26.5 mm diameter):
Pre-stress force = 200 kN (100 kN for each bar)
Beam and column cross-section size = 400(d) × 244(b) mm
Steel plate does not cover the full cross-section of member.
Plate size = 350(d) × 200(b) × 32(t) mm
Effective area in beam and column interface = 84,800 mm^2
Effective area in column at steel plate interface = 57,200 mm^2

Average stress in beam = 2.36 N/mm^2
(Corresponds to 5.24 % of LVL compressive parallel to grain strength)
Average stress in column at beam interface = 2.36 N/mm^2
(Corresponds to 20 % of LVL compressive perpendicular to grain strength)
Average stress in column at steel plate interface = 3.50 N/mm^2
(Corresponds to 29 % of LVL compressive perpendicular to grain strength)

d. Important summary of experiment

Experiment is in the setting up stage. No summary.

e. Progress to date

Experiment is in the setting up stage.

f. <u>Important Lessons</u>

1. After 1 year of service life, post-tensioning losses up to 50% have been estimated in the frame with unreinforced column connections compared to 10% loss in the frame with fully armoured columns.

Experiment 5 – Long-term monitoring of post-tensioned shear walls in NMIT building

a. Background

This experiment is being carried out by Hugh Morris, Senior Lecturer at the University of Auckland (Morris et. al., 2010; Morris et. al., 2011a; Morris et. al., 2011b), to monitor the long-term deformations of shear walls and floors of the Nelson Marlborough Institute of Technology (NMIT) Arts and Media building. The building has four pairs of post-tensioned LVL shear walls 189mm (t) × 3m (w) × 12m (h) in both directions (Figure 5.1). The post-tensioning steel in each wall consists of four 32 mm diameter Macalloy bars. Initial post-tensioning loads of 348 kN per bar were applied in June 2010 while accurate instrumentation was set up in April 2011.

Local deformations have been observed at the top of the walls, just under the bearing plate for the post-tensioning.

The shear walls are linked to the frame and lateral loads transferred through the concrete floor diaphragm. In the event of an earthquake, the shear walls are designed to rock side to side with U-shaped flexural plate energy dissipating devices between each pair of shear walls. The majority of the upper floor area uses the non-composite "Potius" proprietary LVL system with a non-structural concrete topping, while the main floor beams have shear connectors to provide composite action with the concrete slab along the main span direction.

b. Objective

The objective of this experiment was to investigate post-tensioning losses in the shear walls due to parallel to grain deformation of the LVL. A parallel experiment (not reported here) is investigating the long-term deformation of the floors and beams (composite and non-composite).

c. Pre-stress Force and Stresses

Wall cross-section = 700 × 189 mm
Steel plate does not cover the full cross-section of member.
Plate size = 600(l) × 125(w) × 50(t) mm
Hole area = 37,800 mm^2
Effective area under plate = 37,200 mm^2
Effective area of wall = 94,500 mm^2

	June 2010 (initial)	Oct 2011 (after 16mths)
Total post-tensioned force (kN)	1387.4	1133.2
Stress on plate	18.5	15.1
Stress directly under the plate on LVL wall (N/mm^2)	37.3 [83%]	30.5 [68%]
Stress at main area wall (assume force is distributed over the full area of wall) (N/mm^2)	14.7 [33%]	12.0 [27%]

Note: [ ] percentage of compressive parallel to grain strength

It is noted that the steel bearing plate could have easily been designed using a larger bearing area, with more wood provided inside the wall cavity and a wider steel plate to cover the full width of the wall.

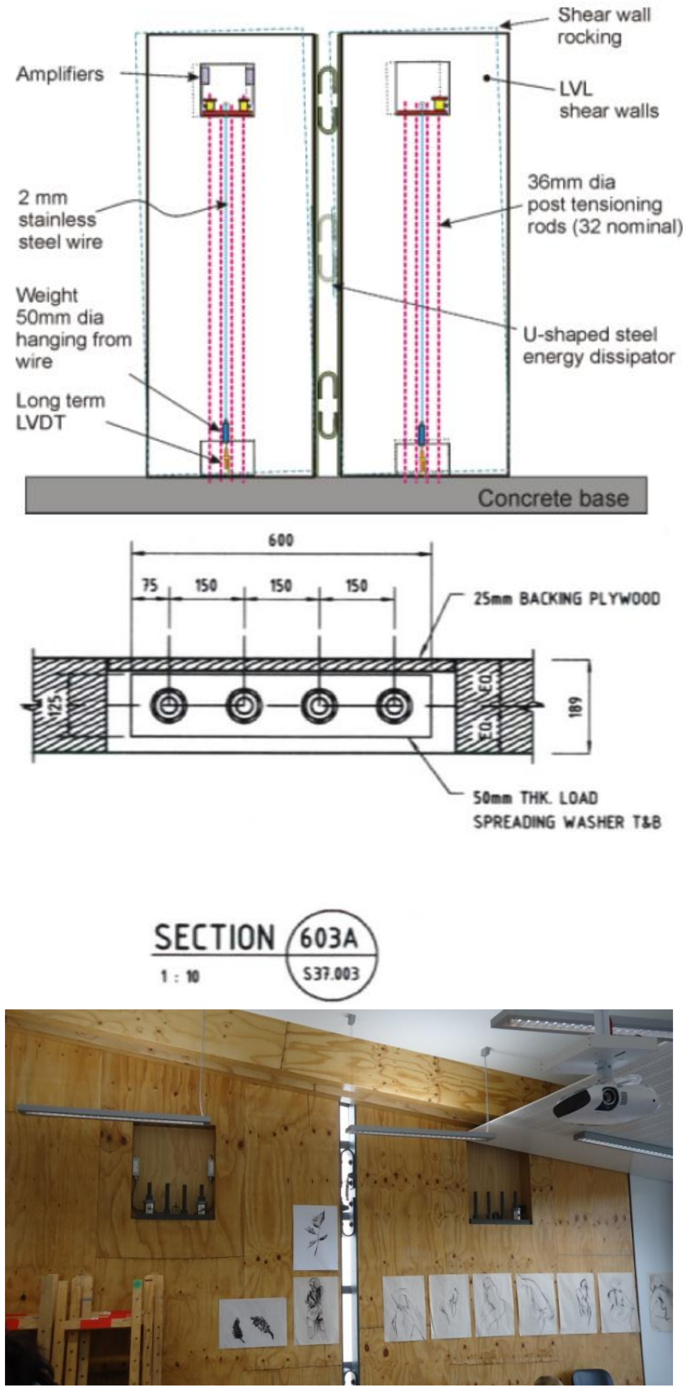


Figure 5.1 Post-tensioned shear walls in the NMIT Arts Building

d. <u>Important summary of experiment</u>

1. Elastic deformation immediately after initial post-tensioning was approximately 2.5 mm based on manual measurement.

2. Over 9 months, the shear walls deformed 0.66 to 1.4 mm under the compressive load of the post-tensioned force.
3. Over 18 months since post-tensioning (June 2010), the tension losses were 13.1 % to 20.4 % (Table 5.1).
4. To date, re-tensioning has not been deemed necessary by the design engineers for the building.

Table 5.1 Change in post-tensioning loads over time for different load cells (Morris et. al., 2011a)

Time	Jun-10 2010	Jun-11 2010	April 2011	June 2011	August 2011	October 2011	December 2011	Losses 18Mths	Losses 18Mths	Losses
kN	Initial	1 day	10 Mths	12 Mths	14 Mths	16 Mths	18 Mths	kN	% Initial	% from day 1
LC3	348	347.6	293	292	286	282	282	65.9	18.9	18.8
LC4	350.5	337.5	317	312	303	302	304	46.1	13.1	9.8
LC5	347.9	344.7	289	286	278	276	277	71.1	20.4	19.7
LC6	341	334.4	289	284	274	273	275	65.9	19.3	17.7

e. Progress to date

This experiment is on-going.

f. Important Lessons

1. Presently the maximum post-tensioning loss is 20.4 %. The structural engineers of the building found that re-tensioning was not yet necessary. However if losses persist, re-tensioning will be inevitable.
2. Careful design of anchorage plates is necessary to prevent high local stresses.

Experiment 6 – Long-term monitoring of post-tensioned frames under variable cyclic environment at UTS (2010 – continuing)

a. Background

These experiments were carried out in University Technology of Sydney under the supervision of Dr. Rijun Shrestha and Professor Dr. Keith Crews (Hallen, 2010; Hallen, 2012; Hallen et. al., 2012).

Specimens include post-tensioned LVL beams for study on parallel to grain effects and post-tensioned LVL frame sub-assemblies comprised of two beams and a column segment for study on perpendicular to grain effects. Screws were introduced as reinforcement into some of the columns to minimise perpendicular to grain crushing, but no steel bearing plate was provided to distribute the load from the screw heads to the ends of the beams. Two sets of each specimen type were made. The first set was tested under ambient environmental conditions in the laboratory and the second set was subjected to cyclic environmental conditions in a climate chamber where for the first 3 months, they were monitored under the ambient environment of the laboratory and later subjected to severe four to six weeks of environmental cycling of up to 100 % relative humidity with temperatures maintained at 20 °C (Figures 6.1 and 6.2).

Figure 6.1 Post-tensioned LVL beam and frame specimens under uncontrolled non-cyclic environment

Figure 6.2 Post-tensioned LVL beam and frame specimens under controlled cyclic environment (left) and compressive perpendicular to grain failure in the column at the beam-column interface (right).

b. <u>Objective</u>

The objective of these experiments was to investigate the long-term performance (post-tensioning losses due to creep in timber parallel and perpendicular to grain) of post-tensioned timber frames (sub-assemblies of beam-column-beam connections) under variable cyclic environmental conditions.

c. <u>Pre-stress Force and Stresses</u>

Pre-stress force = 230 kN (26 mm CT StressBar by VSL)
Beam cross-section = 360(*d*) × 189(*b*) mm
Column cross-section = 300(*d*) × 189(*b*) mm

Effective bearing area = 57,330 mm^2

Average stress in beam = 4.01 N/mm^2
(Corresponds to 8.9 % of LVL compressive parallel to grain strength)

Average stress in column = 4.01 N/mm^2
(Corresponds to 33.4 % of LVL compressive perpendicular to grain strength)

d. <u>Important summary of experiment</u>

1. For the first 3 months under ambient laboratory environment, there was approximately 10 % pre-stress loss in the beam-only specimens and 20 % loss in the frame sub-assembly specimens.
2. After approximately 1 month under cyclic environment, there was obvious compression perpendicular to grain failure at the beam-column interface in the frame sub-assembly specimens.
3. It is recommended to restart tests on these frame sub-assembly specimens with a steel plate between the beam-column interface and the reinforcing screws in the column.
4. It is also recommended to place the beams on their sides to provide lateral stability and eliminate kinking due to local crushing.

e. <u>Progress to date</u>

This experiment is on-going.

f. <u>Important Lessons</u>

1. Some form of reinforcing at the beam-column interface is crucial to prevent local crushing.

Concluding remarks

The following important conclusions can be drawn:

1. It is important to reinforce the column at the beam-column interface to prevent local crushing in compression perpendicular to the grain.
2. The use of steel inserts through the column and a steel plate at the interface were found to be effective to minimise pre-stress losses and local crushing. The use of screws to reinforce the column is another method but was found to be less effective compared to steel inserts without a steel bearing plate. More testing is necessary.
3. Steel bearing plates under the post-tensioning anchorages need to be designed with as large bearing area as possible, and to cover the full width or section of the LVL member.
4. Cavities in the LVL member need to be minimised under the post-tensioning anchorages in order to provide sufficient bearing area.
5. The environmental conditions must be carefully accounted for in the design of the bearing area. Based on Experiment 6, in a severe variable condition (Service Class 2 to 3), failure in a column reinforced with screws occurred when the average stress was only 33.4 % of the compressive strength perpendicular to the grain. In Experiment 2, failure in the anchorage occurred under a Service Class 3 condition when the average stress was only 26 % of the compressive strength parallel to the grain.

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