

Seismic performance of a post-tensioned LVL building subjected to the Canterbury earthquake sequence

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ABSTRACT: The following paper presents the seismic performance of a two storey post-tensioned Laminated Veneer Lumber (LVL) building during the aftershock sequence following the MW 6.3 Canterbury earthquake that occurred on 22nd February 2011. Composed of post-tensioned walls in one direction and post-tensioned frames in the other, the structure under analysis was originally tested quasi-statically in the structural laboratories of the University of Canterbury (UoC), Christchurch, New Zealand. Following testing the building was demounted and reassembled as the offices of the STIC (Structural Timber Innovation Company) research consortium on the UoC campus with several significant changes being made to convert the building from its initial use as a test specimen into a functioning office structure.

Just prior to the start of construction, the MW 7.1, 4th Sept 2010 Darfield earthquake occurred in the Canterbury area, however construction went ahead as planned with the building being almost complete when the more devastating 22 February 2011 event occurred. An array of 3-dimensional acceleration sensors was installed on the building follow completion.

Innovative techniques have been used in the study of the building seismic response allowing clear representation of the buildings dynamic performance. The damage free nature of the building even under its Ultimate Limit State excitation is also clear.

1 INTRODUCTION

Post-tensioned timber construction is an innovative new technology which is currently used in New Zealand in the construction of multi-storey seismic resistant timber structures and is being adopted worldwide. Dynamic structural analysis is an ever-growing research field with innovative methods and technologies being developed continuously.

1.1 *The System*

Recent developments in the field of seismic design have led to the development of damage control design philosophies and innovative seismic resistant systems. In particular, jointed ductile connections for precast concrete structures have been implemented and successfully validated. One jointed ductile connection, originally developed for precast concrete during the U.S.-PRESS program (PREcast Seismic Structural System), coordinated by the University of California, San Diego, for frame and wall systems has been particularly successful (Priestley et al. 1999). This system, referred to as the hybrid system, combines the use of unbonded post-tensioned tendons with grouted longitudinal mild steel bars or any other form of dissipation device

The post-tensioned timber concept has been developed and extensively tested at the University of Canterbury using laminated veneer lumber (LVL), in a system known as Pres-Lam. Over the last seven years extensive medium scale sub-assembly testing has been performed (Palermo et al. 2006). Once the principles of the post-tensioned timber system had been validated, larger scale tests were proposed and performed. The first of these was a full scale internal and external beam-column

connection, (Iqbal et al. 2010), followed by a 2/3rd scale frame and wall assembly test both with and without flooring, (Newcombe et al. 2010). Both of these tests continued to validate the excellent performance of the system. This method of construction has recently also been used in the construction of a series of multi-storey buildings in New Zealand. The first of these was a new structure for the Nelson Marlborough Institute of Technology (NMIT) as described in Devereux et al. (2011).

1.2 The Structure

The structure which was analysed was initially a two storey test structure which was subjected to a series of quasi-static tests in the structural laboratory of the University of Canterbury. The two storey structure was a frame and wall open plan building as shown in Figure 1. The building consisted of lateral resisting post-tensioned timber frames in one direction and post-tensioned shear walls in the opposite direction. The floor of the structure, which was of area 41m² on two levels (i.e. a total area of 82m²), was a timber concrete composite flooring system developed at the University of Canterbury, consisting of LVL joists and 50 mm of topping concrete connected with notch and coach screw discrete couplers (Yeoh 2010). Due to the structure being a research specimen the floors spanned in two different directions, with the flooring spanning in the long (Frame) direction on the lower floor.

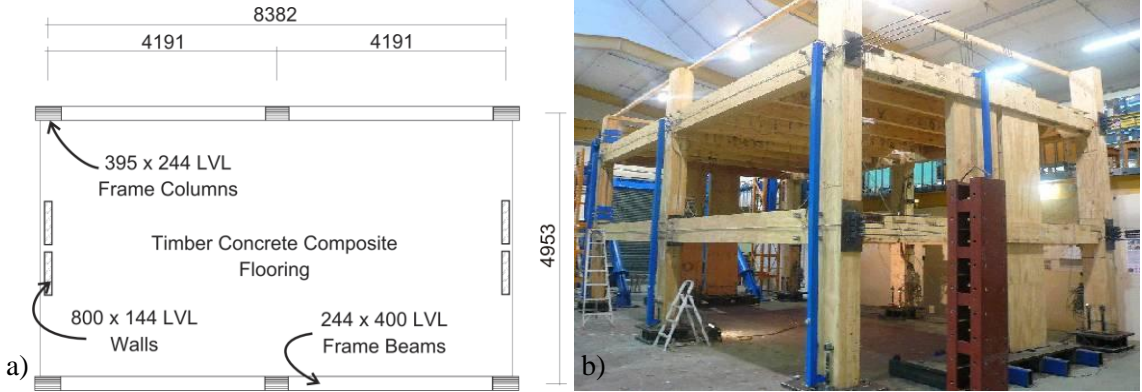


Figure 1. Pres-Lam Test Building a) Floor Plan and b) Constructed Test Building

Quasi-static cyclic testing was performed in both the frame and wall directions separately as well as simultaneously. The test building displayed excellent seismic performance with complete recentering and no significant damage up to 2% drift (Newcombe et al. 2010). It was noted that the simultaneous bi-directional loading had no major effect on the in-plane resistance the frames or walls. Once the Pres-Lam test building was completed, a proposal was made to recycle the structural components to form a new office structure for STIC, the Structural Timber Innovation Company (Fig. 2).



Figure 2. The Expan Office Structure for STIC

To transform the experimental building into a new office building, most components of the existing experimental building would be fully utilised. In total over 90% of the structural components were

able to be reused (Smith et al. 2011), however due to the original purpose as a 2/3 scale test specimen, some changes had to be made. The building was designed before the September earthquake therefore, the codes in place at the time were used in design (Z for Canterbury = 0.22). The building was estimated to have a fundamental period of 0.34 seconds. The building was given an Importance Level of 2 and due to the temporary nature of the building a 10 year design life was assumed ($R_{ULS} = 0.75$).

2 BUILDING INSTRUMENTATION AND SELECTED RECORDS

The structure was almost fully complete when the February 22nd event occurred however instrumentation had not been yet installed. Instrumentation was installed on the Expan building at the end of March 2011 and consisted of three triaxial accelerometers mounted at the foundation, first floor and second floor. As shown in Figure 3, the first and second floor accelerometers were positioned in the centre of the central beam, while the foundation accelerometer is near the west end of the structure.

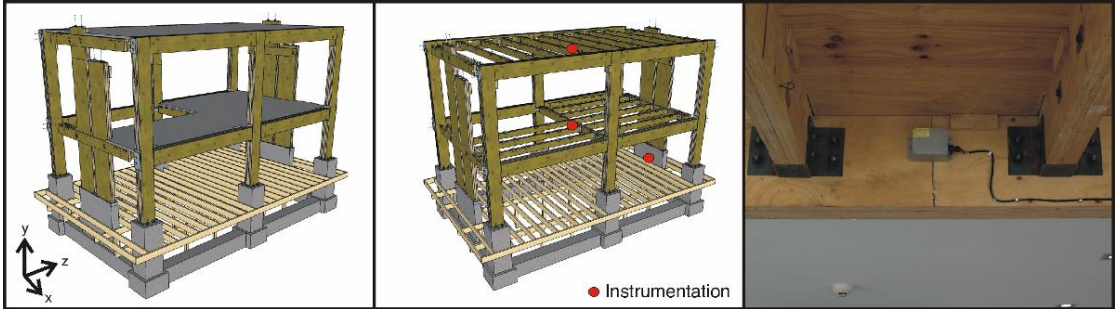


Figure 3. Instrumentation layout placed on the Expan building, showing axes for recorded accelerations. Note the varying purlin layout at each level, and the irregular concrete diaphragm.

The three instruments represent a full CUSP-3C3 unit with two external sensors. Response data is captured by each accelerometer, then calibrated by hardware on-board each module before output. This raw data is then collected by hardware custom to the CUSP-3X system, which interfaces with a Linux based computer to log data. Data is only captured during seismic activity, with a 20 second buffer either side of a threshold-triggering event.

Following the installation of the instrumentation trigger thresholds have been surpassed over 1000 times leading to a significant database of records available to the current research. In this paper a selection of 6 records have been chosen based on the largest acceleration measured at the second storey of the building between the installation of the instrumentation and the 1st of October 2011. Information regarding the selected records is presented in Table 1 and shown in Figure 4. Accelerations shown refer to the records at the building foundation in the Z (frame) direction.



Figure 4. Studied Earthquakes in Relation to Expan Structure (NASA Satellite Image)

Table 1. Selection of Earthquakes used in Study (sourced from Quake Search - The Earthquake commission and GNS Science)

	Mag.	Depth (km)	Date and Time (CUT)		Date and Time (Local)		Acc. Z (g) Foundation
A	6.41	6.92	13/6	2:20 am	13/6	2:20 pm	0.393
B	5.89	8.90	13/6	1:01 am	13/6	1:01 pm	0.437
C	5.54	9.33	5/6	9:09 pm	6/6	9:09 am	0.279
D	5.44	8.67	21/6	10:34 am	21/6	10:34 pm	0.285
E	5.31	8.96	16/4	5:49 am	16/4	5:49 pm	0.173
F	5.24	12.00	9/5	3:04 pm	10/5	3:04 am	0.255

3 STRUCTURAL PERFORMANCE

A series of indicators have been used to monitor building performance during the seismic sequence described. These range from simple visual inspection to advanced techniques including as S-transform analysis of acceleration data. Two techniques were also used to evaluate damping, which will also be presented. This section describes and analyses the results of these monitoring data evaluations

3.1 Visual Inspection

While the Expan building was not constructed during the September 2010 earthquake, it was 95% completed when subject to the February 2011 earthquake. The only building components not installed in February 2011 were the spiral staircase and the railing around the opening of the 2nd floor. Extensive visual inspections verified that the building suffered no damage to the structure, the interior linings or the exterior cladding during the February 2011 event. Additionally, subsequent aftershocks and the earthquakes in June (shown in Figure 6) and December 2011 have also not resulted in any damage to any of the building components or the structure.

3.2 Time History Accelerations

The first study made of the data involved the evaluation of the individual time history responses. One of the principle objectives of this was to ensure instrumentation was functioning properly and to study the way in which accelerations were transmitted up the structure.

Figure 5 and Figure 6 show the time-history response of the building under the largest recorded excitation for the period selected (A, the 13th of June Event). Along with the full time-history, a window has been shown between 16 and 17 seconds in order to show clearly the dynamic response of the building. It can be seen that a significant increase in acceleration occurs from the base of the structure to the 3rd floor with this effect being larger in the Z (frame) Direction.

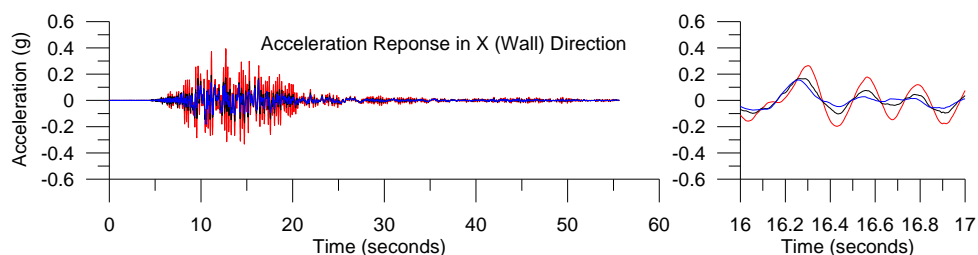


Figure 5. Time History Response in X (Wall) Direction for Earthquake A – 13th June 2011 2:20 pm (Blue – Ground, Black – First Floor, Red – Second Floor)

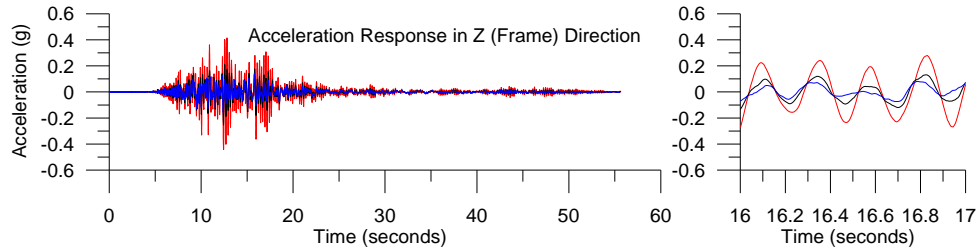


Figure 6. Time History Response in Z (Frame) Direction for Earthquake A – 13th June 2011 2:20 pm (Blue – Ground, Black – First Floor, Red – Second Floor)

3.3 Spectral Response

The acceleration spectra for the six selected earthquakes are shown in Figure 7. Along with the recorded responses the thick red line shows the design spectrum used and derived from the design parameters stated in Section 1.2.

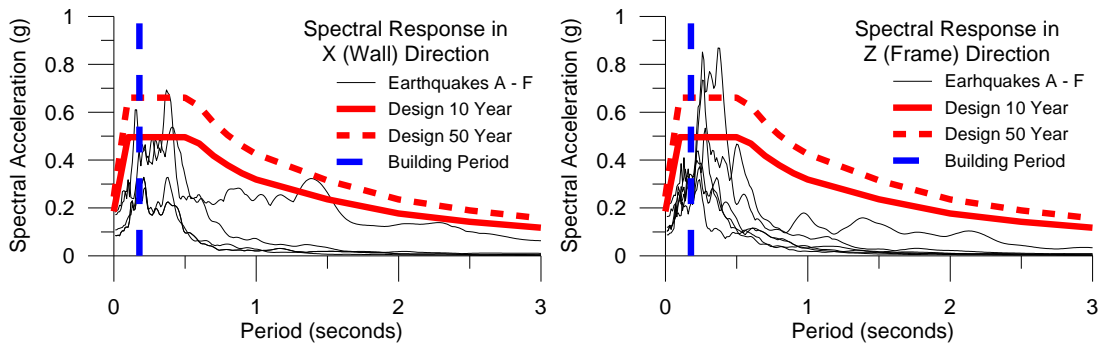


Figure 7. Acceleration Spectra for Selected Records

Direct comparison between the input spectra and design spectrum shows that on more than one occasion input has been either equal to or above the 10 year design value and in one case approaches the 50 year design value in the range of the building period (calculated to be 0.18 as shown in Section 3.5 and shown as a blue dashed line in Figure 7).

3.4 Damping

Two methods were used in order to evaluate the equivalent viscous damping of the Expan Building. The first of these (Ewins 1994) was the half-power bandwidth (HPB) method, a method which estimates the damping using the frequency range, in combination with a Welch Fourier analysis (Welch 1967). Generally, the HPB method returns significant results in the analysis of a stationary system, however, in this case the method was applied also using a forced motion recording (i.e. non-stationary) in order to compare results obtained with those of the second method described below that is more appropriate under forced conditions. The HPB method was applied using the following steps for both horizontal components: selection of the accelerometric recording related to the selected direction, baseline removal, trend removal, and evaluation of the Welch Spectrum.

In the application of the HPB method, firstly, the amplitude of each (in this case the first) natural frequency is obtained. Two more frequencies are then taken that have an amplitude equal to the amplitude of the natural frequency divided by the square-root of two. The distance between these two points will increase as damping increases thus damping is taken as the ratio between these points.

The second method was as proposed by Mucciarelli and Gallipoli (2007) for the simple non-parametric analysis (NonPaDAn) of the damping factor of buildings. This simple method allows the calculation of damping values from a single short input, also under forced conditions, using statistical analysis of decreasing peaks in the displacement, velocity, or acceleration time history response. The

damping factor is estimated using the logarithmic decrement method on a minimum of three consecutive decreasing peaks separated by the same period T (within a bracket of \pm the tolerance level (ϵ) as a function of T). The values obtained using the two methods are shown in Figure 8 which shows the damping in the X (black squares) and Z (red dots) along with the average of the values (red and black continuous lines for X and Z direction respectively).

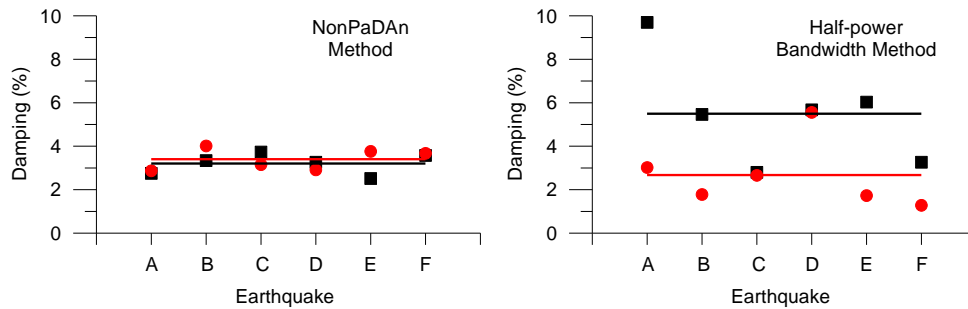


Figure 8. Damping of Expan Building Under Excitation

The above figure shows that the NonPaDAn method is more stable than the HPB method as it is more appropriate for use under the given conditions (i.e. a forced system situation). The results of the NonPaDAn method show nominal damping in the system of between 3 and 4%. It should be noted however that this value is an average of damping obtained throughout the full seismic event. It is therefore possible that damping increases above this range during strong motion. Work is ongoing in the investigation of damping over time.

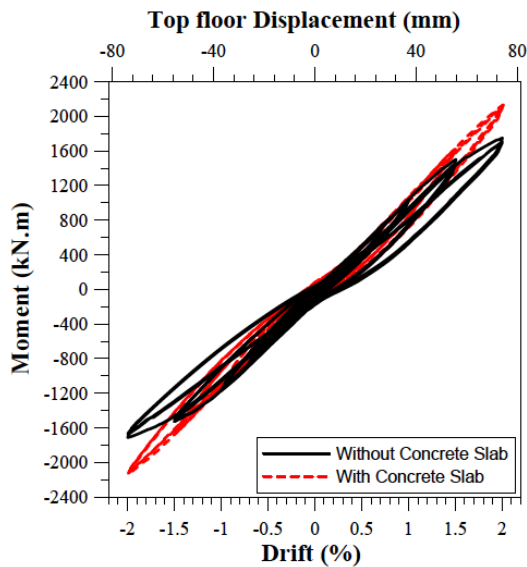


Figure 9. Force-displacement of Pres Lam Test building under Quasi Static Excitation (Frame Direction)

behaviour) is not adequate for the study of a system which changes its characteristics over time.

In order to overcome some of the inadequacies of the Fourier Transform several methods have been proposed such as the Short Time Fourier Transform (STFT) (Gabor, 1946), Wavelet Transform (Daubechies, 1992) or the Wigner-Ville Distribution (Wigner, 1932; Ville, 1948). However all of these methods have limitations which restrict their usefulness in the analysis of non-stationary signals (Ditommaso et al. 2012).

A tool that overcomes the limitation of the previously described methods is the S-Transform (Stockwell et al., 1996). This transform allows the accurate assessment of both the spectral

characteristics and their local variations over time. For a signal $h(t)$ the S-Transform is described as:

$$S(\tau, f) = \frac{|f|}{2\pi} \int_{-\infty}^{+\infty} h(t) \cdot e^{-\frac{(\tau-t)^2 \cdot f^2}{2}} \cdot e^{-i \cdot 2 \cdot \pi \cdot f \cdot t} dt \quad (1)$$

Where: t = time, f = frequency and τ = a parameter that controls the position of a Gaussian window along the time axis. This method of analysis has been used on the selected records with the results of the analysis of Earthquake A shown in Figure 10. In the figure, the output of the S-Transform has been normalized in order improve clarity.

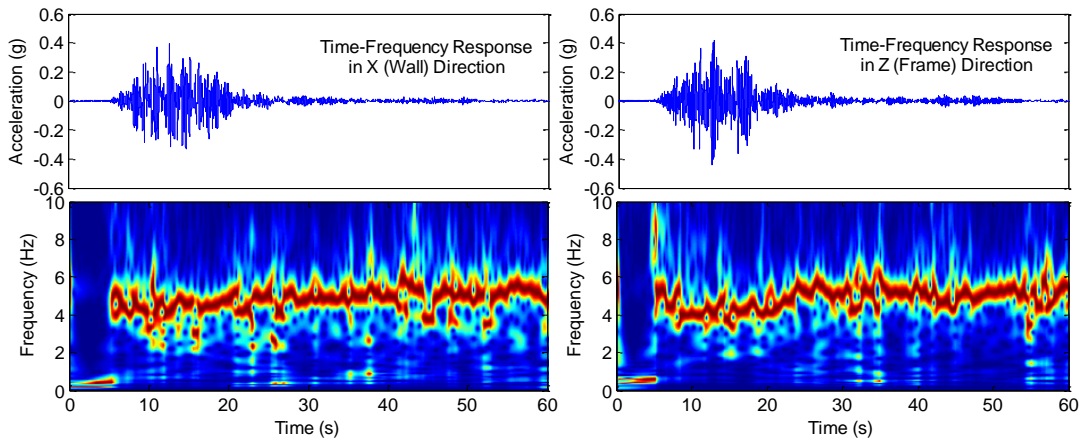


Figure 10. S-Transform of Earthquake A in X and Z direction given Time Frequency Response

The first thing that can be noted from Figure 10 is that the fundamental frequency of the building in both directions is approximately 5.5 Hz which gives a fundamental period of 0.18 seconds. This is lower than the estimated value which was used in design ($T = 0.34$ s). The closeness of the two values indicates that probably a torsional mode is governing the system response. This is possible and may be arising from the large section of floor which was removed in order to allow for the stairs (Fig. 3.)

Figure 10 also shows the nature of the non-linear elastic system. More evident in the Z direction than the X direction, there is a clear drop in frequency (to around 4 Hz or 0.25 s) during excitation which is fully recovered following the seismic event. If this analysis is performed on a system which is damaged during the event the frequency of the structure will not return to its original value but instead to a third 'post-event' value (Ponzo et al. 2010). This is not the case however as the original stiffness of the structure is regained as the building returns to its original position damage free.

4 CONCLUSIONS

The seismic response of a post-tensioned timber building has been studied and the preliminary results have been presented. The structure, which began life as a laboratory test specimen, has been constructed as the offices of the Structural Timber Innovation Company (STIC) on campus at the University of Canterbury and named the Expan Building. The structure was 95% completed when the earthquake of February 2011 struck and has since been subjected to subsequent aftershocks without any damage to building components or the structure.

Three strong-motion sensors were installed on the structure in March 2011 and have registered over 1000 seismic events since activation. The largest six events have been considered for this study and have been shown to be near and in some cases to exceed considered design values (considering that structural design took place before September 2010).

The time histories of these records have been studied and show that a significant increase in acceleration is registered from ground input to the second storey with this effect being more severe in the frame direction. All signals were analyzed using both a standard approach, based on the response

spectra, and an innovative approach based on the S-Transform. This latter approach allows the analysis of the time-varying behaviour of the building in the time-frequency domain. Study of the time-frequency response of the structure shows the non-linear elastic nature of the system with the fundamental period returning to its base value of 5.5 Hz (0.18 s) following even the strongest event.

In order to compare the results, two methods of damping calculation were used. The NonPaDAn method, more adept at calculating damping under forced excitation provided average results of 3 – 4% consistent with laboratory test results. Further analyses are necessary to better understand the non-stationary behaviour of the Expan Building and to better understand the dissipative behaviour of post-tensioned timber buildings during strong motion excitation.

5 ACKNOWLEDGEMENTS

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