The Seismic Performance of a Post-tensioned LVL Building During the 2011 Canterbury Earthquake Sequence

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SUMMARY:
The seismic performance of a two storey post-tensioned Laminated Veneer Lumber (LVL) building during the aftershock sequence following the 6.3 M_W 22^{nd} February 2011 Canterbury earthquake is presented. The building is made from a new form of timber construction combining the use of concrete PRESSS technology and wood products using post-tensioning elements with large timber members. Originally a test specimen, the building was demounted and reassembled as the offices of the STIC research consortium on the campus of UoC. Close to the beginning of construction the 7.1 M_W 2010 Darfield earthquake occurred in the Canterbury area however construction went ahead as planned with the building being almost complete when the more devastating 2011 February event occurred. Innovative techniques have been used to evaluate the seismic response of the building and this paper presents a general overview of building performance and provides insight into the behaviour of a post-tensioned structure.

Keywords: 2011 Canterbury Earthquake, Post-tensioned timber, Earthquake Monitoring, Multi-storey Timber Buildings

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. This paper looks into the use of monitoring techniques in the analysis of the seismic performance of post-tensioned timber technology.

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.-PRESSS 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 (UoC) 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 consistent and predictable 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) and other buildings have followed (Buchanan et al. 2011).

1.2. The Structure

The structure which was analysed was initially a two storey test structure (the Pres-Lam test building) 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 1.1. 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 41 m^2 on two levels (i.e. a total area of 82 m^2), was a timber concrete composite 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.

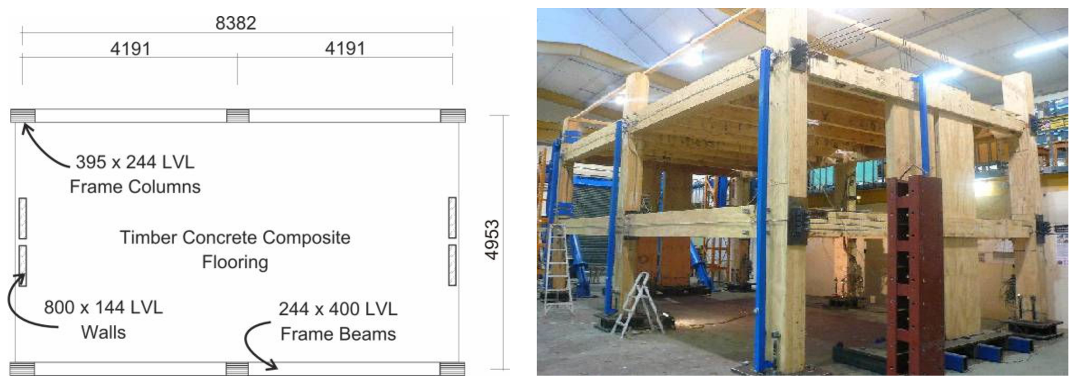


Figure 1.1. Pres-Lam Test Building a) Floor Plan (distances in mm) and b) Constructed Test Building

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 of the frames or walls. Once the testing of 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 (Figure 1.2) and called the Expan Building.

Figure 1.2. The Expan Office Structure for STIC

To transform the experimental building into a new office building, most components of the existing experimental structure were to 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 2010 earthquake therefore, the codes in place at the time (NZS 1170.5:2004 2004) were used in design (Z for Canterbury = 0.22). The building was estimated to have a fundamental period of 0.34 seconds and 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 2.1, the first and second floor accelerometers were positioned in the centre of the central beam, while the foundation accelerometer was placed near the west end of the structure.

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.

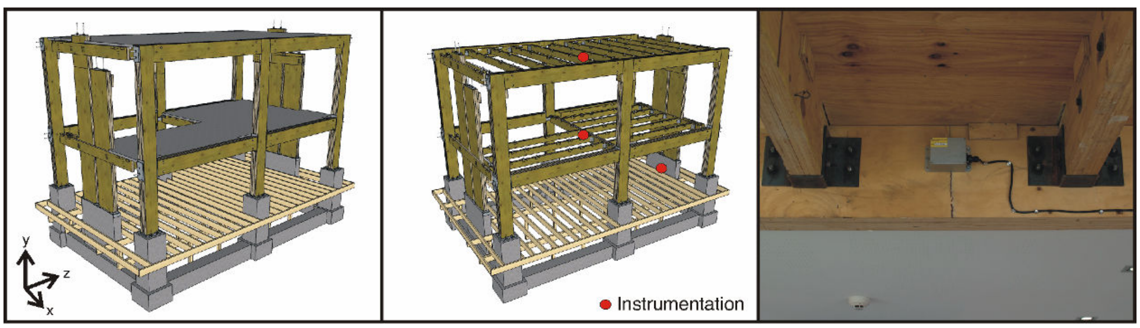


Figure 2.1. Instrumentation Layout Placed on the Expan Building, Showing Axes for Recorded Accelerations. Note the Varying Purlin Layout at Each Level, and the Irregular Concrete Diaphragm. Far Right Shows Installed Accelerometer.

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 2.1 and shown in Figure 2.2.

Table 2.1. 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)*
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

*Accelerations shown (Acc. Z) refer to the records at the building foundation in the Z (frame) direction.

Figure 2.2. Studied Earthquakes in Relation to Expan Structure (NASA Satellite Image)

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 S-transform analysis of acceleration data. The Housner Intensity (Housner 1952) has been calculated and compared with the values of spectral acceleration, drift and Peak Ground Acceleration (PGA). Damping has been evaluated using the NonPaDAn (Mucciarelli and Gallipoli 2007) method. 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 subjected to the February 2011 event. 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 (Earthquake A in Figure 2.2) and December 2011 have also not resulted in any damage to any of the building components or the structure.

3.2. Time History Response

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 3.1 shows the acceleration, displacement and drift recorded during Earthquake A.

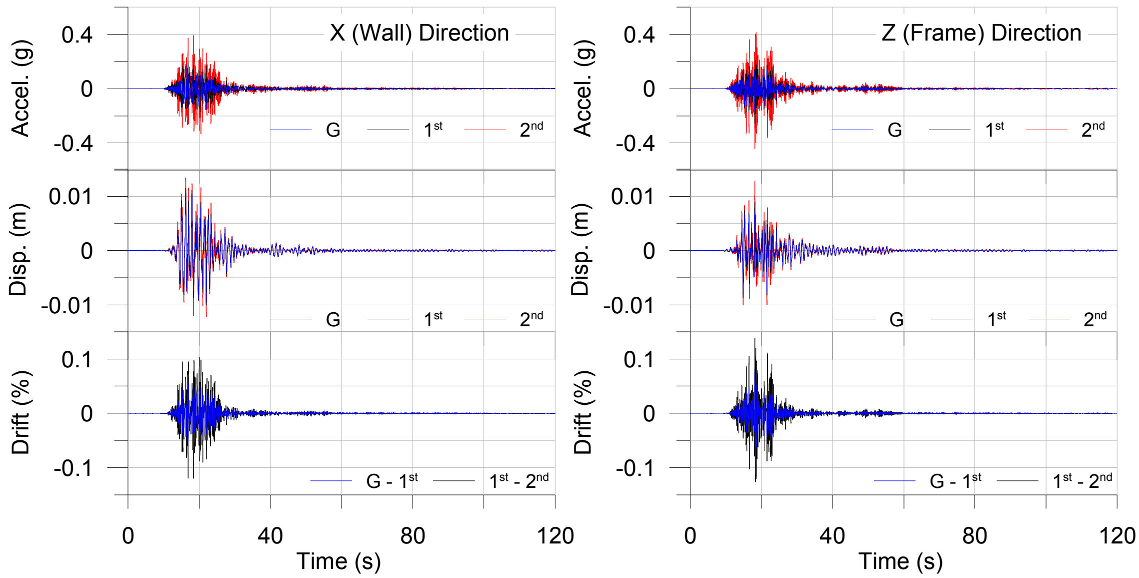


Figure 3.1. Time History Response In X (Wall) And Z (Frame) Direction In Terms Of: Acceleration, Displacement And Drift

From the figure showing the acceleration time-history it can be seen that a significant increase in acceleration occurred from the base of the structure to the 2nd floor (roof) with this effect being larger in the Z (frame) direction. Maximum interstorey drifts recorded were not significant enough to observe the elastic non-linear behaviour which is characteristic of this type of construction system. Interstorey drifts were not constant throughout the structure with larger values occurring at the second level. This is likely due to the large concrete plinths and effectively fixed base connection which were used as shown in Figure 2.1.

3.3. Spectral Response

The acceleration spectra for the six selected earthquakes are shown in Figure 3.2. Along with the recorded responses the red line shows the design spectrum used as derived from the design parameters stated in Section 1.2.

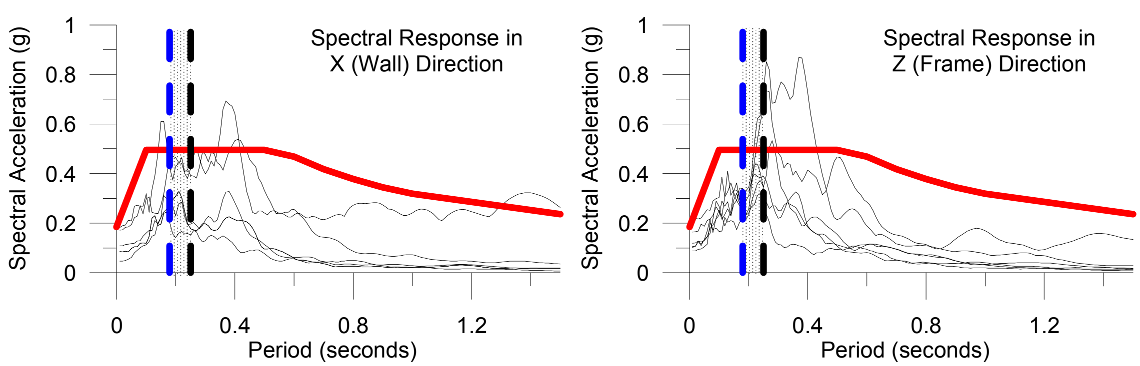
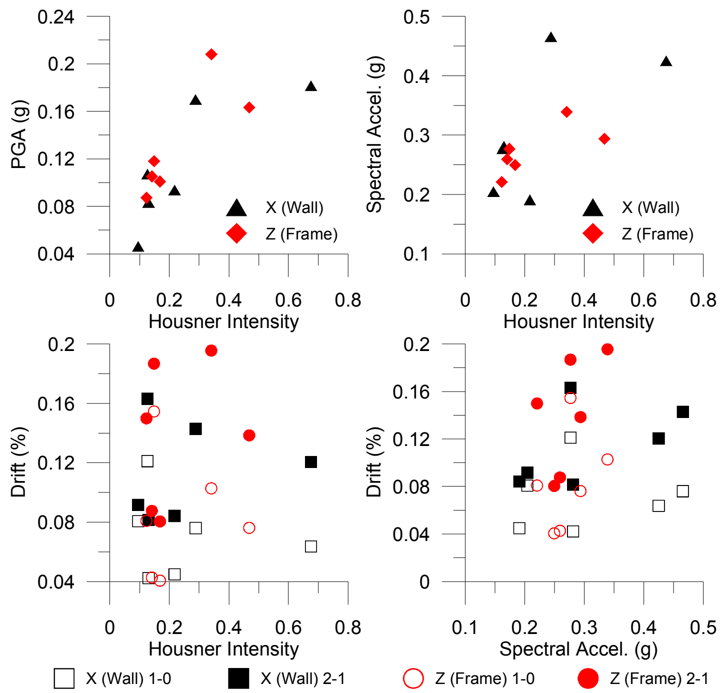


Figure 3.2. Acceleration Spectra for Selected Records

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 design value in the range of the building period. This has been calculated to be 0.18 in both directions which has been identified as a possible torsional mode (shown as a blue dashed line in Figure 3.2). The black dashed line and shaded area indicates the frequency range during forced motion.

3.4. Examined Correlations

During the examination of the results obtained from the 6 acceleration records being studied several key parameters which are commonly used in structural dynamic analysis have been calculated and compared. These parameters were: Peak Ground Acceleration (PGA), interstorey drift, spectral acceleration at the fundamental period of T = 0.18 seconds and the Housner Intensity. Often used as an indicator for structural damage, the Housner Intensity is evaluated as the area under the velocity spectrum between a given period range (Housner 1952). Comparisons between a selection of these parameters are shown in Figure 3.3 along with a table showing the calculated linear regression analysis values.



Linear Regression R^2 Value	
X Direction	
Housner - PGA	0.67
Housner – Spec A	0.45
Housner – Drift 0-1	0.04
Housner – Drift 1-2	0.03
Spec A – Drift 0-1	0.01
Spec A – Drift 1-2	0.29
Z Direction	
Housner - PGA	0.65
Housner – Spec A	0.50
Housner – Drift 0-1	0.00
Housner – Drift 1-2	0.07
Spec A – Drift 0-1	0.13
Spec A – Drift 1-2	0.30

Figure 3.3. Comparison Between: PGA, Spectral Acceleration, Drift And Housner Intensity and Drift and Spectral Acceleration

As shown Figure 3.3 minimal correlation was found for the 5 cases studied between the Housner intensity and both PGA and Spectral acceleration. No relation was seen however between the Housner Intensity and interstorey drift. The final comparison made was between the spectral acceleration and interstorey drift which also shows no correlation.

3.5. S-Transform

In recent times several techniques for both signal analysis and structural dynamic identification have been proposed in order to characterise the dynamic behaviour of structures (Ditommaso et al. 2012). Most of these techniques are useful in the characterisation of stationary structural behaviour but are not effective when structures display non-stationary and/or non-linear behaviour. One of the most common tools used in the dynamic analysis of systems is the Fourier transform. However, this technique (along with all techniques which are founded on the assumption of stationary system behaviour) is not adequate for the study of a system with changing 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) and 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 limitations 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:

$$S(\tau, f) = \frac{|f|}{2\pi} \int_{-\infty}^{+\infty} h(t) \cdot e^{-\frac{(\tau - t)^2 \cdot f^2}{2}} \cdot e^{-i \cdot 2 \cdot \pi \cdot f \cdot t} dt \tag{3.1}$$

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 top floor acceleration shown in Figure 3.4.

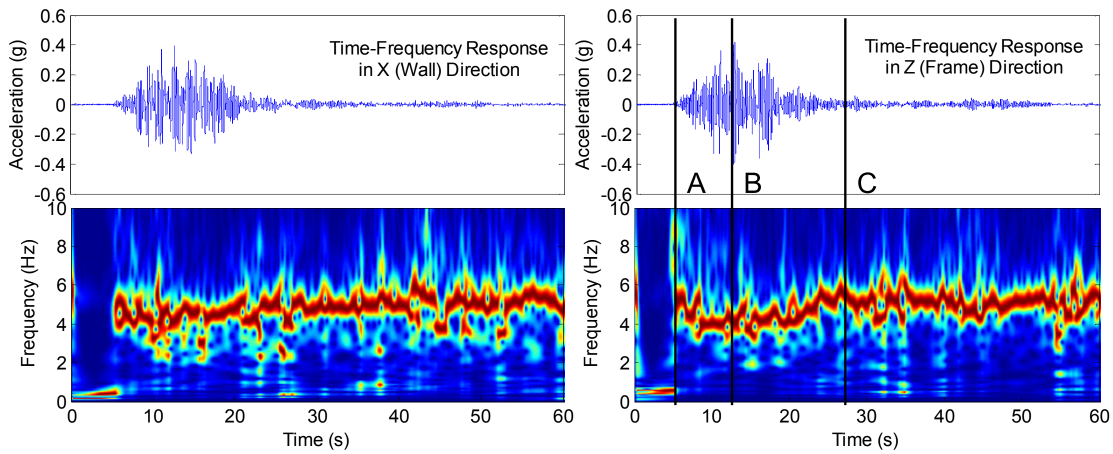


Figure 3.4. S-Transform of Earthquake A in X and Z direction given Time Frequency Response

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 (Figure 3.4).

From Figure 3.4 the relationship between building frequency and drift is shown clearly with the frequency drop from its stationary value of 5.5 Hz to a value of approximately 4 Hz during the maximum excursions of the structure in terms of drift. This is further illustrated in Figure 3.5 which shows the instantaneous base shear versus displacement response at the instants A, B and C in the Z (frame) direction as shown in Figure 3.4. These instances represent the record before, during and post event (A, B and C, respectively). Although the scale of the y axis In Figure 3.5 differs, the ratio between the x and y axis has been maintained.

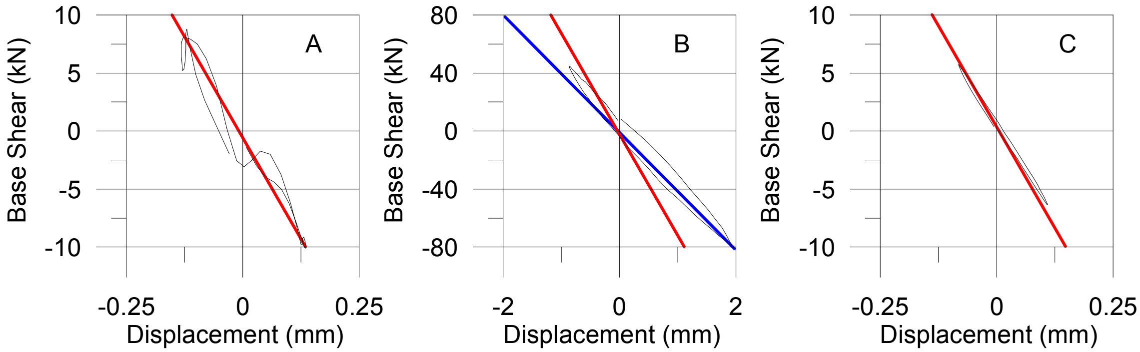


Figure 3.5. Base Shear versus Displacement at Points A, B and C

Figure 3.5 further highlights the way in which the system returns to its initial stiffness (and thus natural period) following a seismic event. This is illustrated by the red and blue lines representing the initial and maximum secant stiffness respectively. As previously mentioned however the maximum displacements reached during the event were not sufficient to induce gap opening and thus the change in stiffness was likely due to slight changes in the dynamic behaviour of the non-structural components. The low level displacements which occurred under what can be considered significant levels of acceleration were due to the systems lightweight nature.

3.6. Damping

Damping was evaluated using a method 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 ± the tolerance level (ε) as a function of T). Damping values obtained for the structure are shown in Figure 3.6 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). The results of the NonPaDAn method show nominal damping in the system of between 3 and 4%. This is reinforced by the base shear versus displacement plots also displayed in Figure 3.6.

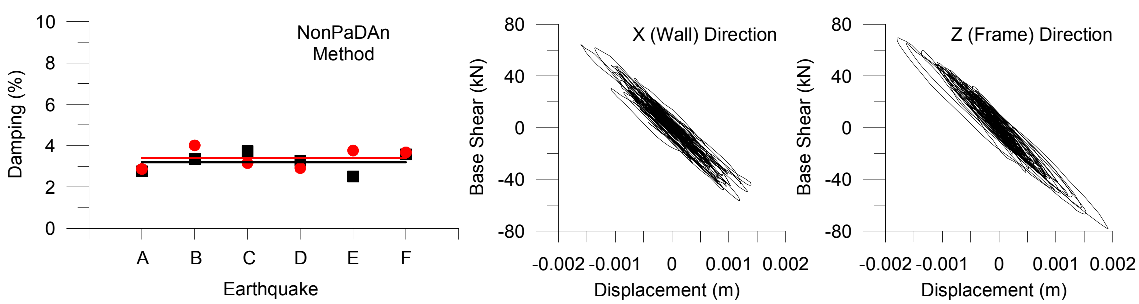


Figure 3.6. Damping of Expan Building under Excitation and Force-Displacement plots for Earthquake A

Although as mentioned the small levels of displacement seen in the structure would indict the absence of significant non-linearity and thus hysteretic damping in the structure results are congruent with the laboratory testing of Newcombe et Al. (2010) which displayed only nominal damping up to design drift levels under quasi static loading.

With this in mind however it is possible to see in Graph B of Figure 3.5 that slight hysteretic damping exists and using the area based method found in Chopra (2001) a value of equivalent viscous damping

(EQV) ζ_{EQV} = 7.2% was calculated. This value represents the EQV of this single cycle which is clearly higher that the value obtained using the NonPaDAn method. This is due to the fact that the statistical NonPaDAn method averages out the damping over the length of the building response. In Figure 3.7 the NonPaDAn evaluation has been performed on selected windows of the building top floor acceleration response in the Z (frame) direction. From this graph the increase in damping during larger displacement cycles can be seen with the method returning a value of ζ_{shock} = 4.2 % during the main shock and ζ_{tail} = 2.6 %; both values considerably less than the ζ_{EQV} value of 7.2% evaluated during the largest displacement response cycle. This is to be expected as the ζ_{EQV} value represents a single maximum point where as the NonPaDAn value of ζ_{shock} and ζ_{tail} represents an average value throughout the total time history response of the building.This relationship between damping and displacement for a post-tensioned timber system without the presence of additional energy dissipating devices was also described in Pino et al. (2010).

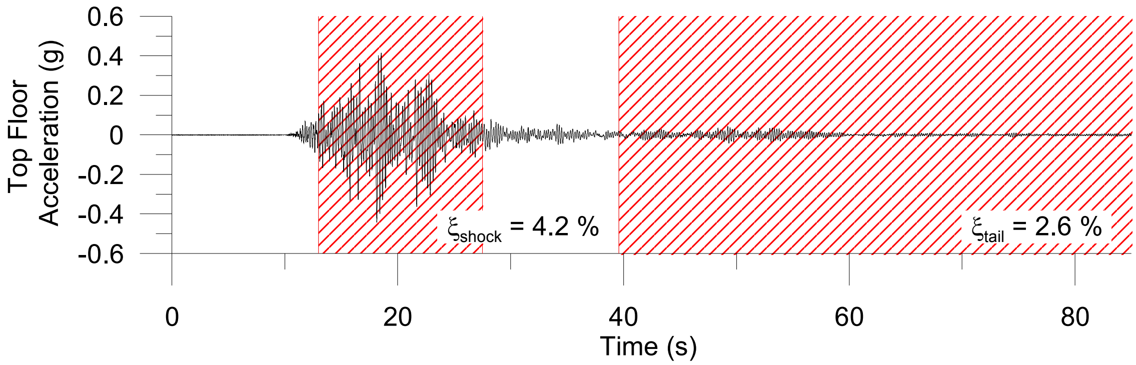


Figure 3.7. NonPaDAn Evaluation on Selected Windows of Top Floor Z (Frame) Direction Acceleration Response

4. CONCLUSIONS

The seismic response of a post-tensioned timber building has been studied and the preliminary results 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. The Housner intensity was calculated and slight correlation was found for the 6 cases studied between the Housner intensity and both PGA and Spectral acceleration. No relation was seen however between the Housner Intensity and interstorey drift. 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 showed that the natural frequency of the building dropped from a stationary value of 5.5 Hz to approximately 4 Hz in correspondence with the building maximum drift. Several instances were extracted which further supported the fact that the structure returns to its initial stiffness following a seismic event.

The damping of the structure has been analysed using the statistical NonPaDAn method which provided an average value of 3% in both building directions. Upon further analysis a relationship between building displacement and damping value was demonstrated with a value of ζ_{EQV} = 7.2 % being calculated for the maximum displacement cycle, ζ_{shock} = 4.2 % during the main shock and ζ_{tail} = 2.6 % during the end of excitation. These values of damping are nominal as the building is without the addition of damping devices leading to significant increases in acceleration up the structure.

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REFERENCES

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

Chopra, A. K. (2001). Dynamics of Structures: Theory and Applications to Earthquake Engineering, Prentice-Hall, Upper Saddle River, NJ, USA.

Daubechies I. (1992). Ten Lectures on Wavelet. Society for Industrial and Applied Mathematics. ISBN 0898712742.

Devereux, C. P., Holden, T. J., Buchanan, A. H., and Pampanin, S. (2011). NMIT Arts & Media Building - Damage Mitigation Using Poat-Tensioned Timber Walls. *9th Pacific Conference on Earthquake Engineering*, Auckland, New Zealand.

Ditommaso R., Mucciarelli M., Ponzo F. C. (2012). Analysis of Non-stationary Structural Systems by Using a Band Variable Filter, *Bulletin of Earthquake Engineering*. DOI: 10.1007/s10518-012-9338-y.

Housner GW. (1952). Intensity of ground motion during strong earthquakes. Second technical report. August 1952, California Institute of Technology, Pasedena, California

Iqbal, A., Pampanin, S., Palermo, A., and Buchanan, A. H. (2010). Seismic Performance of Full-scale Post-tensioned Timber Beam-column Joints. *11th World Conference on Timber Engineering*, Riva del Garda, Trentino, Italy.

Gabor, D. (1946). Theory of communications. *J. Inst. Electr. Eng*, **93**, 429–457.

Mucciarelli M. and Gallipoli M. R. (2007). Non-parametric analysis of a single seismometric recording to obtain building dynamic parameters. *Annals of Geophysics*, **50(2)**, April 2007.

Newcombe, M. P., Pampanin, S., and Buchanan, A. H. (2010). Global Response of a Two Storey Pres-Lam Timber Building. *2010 New Zealand Society for Earthquake Engineering Conference*, Wellington, New Zealand.

Palermo, A., Pampanin, S., Fragiacomo, M., Buchanan, A. H., and Deam, B. L. Innovative Seismic Solutions for Multi-Storey LVL Timber Buildings. *9th World Conference on Timber Engineering*, Portland, U.S.A.

Pino, D., Pampanin, S., Buchanan, A., Carradine, D., and Deam, B. L. (2010). Shake Table Response of Multi-Storey Post-Tensioned Timber Buildings. *2010 New Zealand Society for Earthquake Engineering Conference*, Wellington, New Zealand.

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

The Earthquake commission and GNS Science (n.d) Quake Search Available at: www.geonet.org.nz

Smith, T., Wong, R., Newcombe, M., Carradine, D., Pampanin, S., and Buchanan, A. (2011). The Demountability, Relocation and Re-use of a High Performance Timber Building. *9th Pacific Conference on Earthquake Engineering*, Auckland, New Zealand.

Standards New Zealand. (2004). "Structural Design Actions Part 5 : Earthquake Actions - New Zealand." Secondary Structural Design Actions Part 5 : Earthquake Actions - New Zealand, Standards New Zealand, New Zealand.

Stockwell, R. G., L. Mansinha, and R. P. Lowe (1996). Localization of the complex spectrum: the S transform. *IEEE Trans. Signal Process*., **44**. 998–1001.

Ville J. (1948). Theorie et applications de la notion de signal analytique. *Cables et Transmissions*, **2A**, 61.

Wigner E. (1932). On the Quantum Correction For Thermodynamic Equilibrium. *Phys Rev*, **40**, 749-759.

Yeoh, D. (2010). Timber-Concrete Composite Floor System, University of Canterbury, Christchurch, New Zealand.