Dynamic performance assessment of a multi-storey timber building via long-term seismic monitoring and model updating

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ABSTRACT: The Nelson Marlborough Institute of Technology Arts building is the world's first commercial implementation of a post tensioned Laminated Veneer Lumber (LVL) shear wall system based on EXPAN technology. In order to understand the response of the building in a serviceability level seismic event, System Identification using dynamic monitoring data, Finite Element (FE) modelling, model updating and time history analysis was performed. Several modal frequencies, damping ratios and mode shapes were identified form the seismic response records. Model updating estimated a 16% increase in the stiffness of LVL structural elements, an approximate 90% decrease in concrete, and nearly nil contribution of cladding to stiffness. An estimation of the serviceability response was made based on a single degree of freedom system time history analysis. It was concluded the NMIT building performs well under serviceability seismic loading. The maximum deflection of a 1/25 year event was estimated to be 8mm.

1 INTRODUCTION

In early 2011, the Nelson Marlborough Institute of Technology (NMIT) Arts building was opened as the world's first commercial implementation of the post tensioned timber shear wall system (Devereux et al. 2011) conceived at the University of Canterbury. Because this is the first building of its type to be constructed in the world, the response due to service actions is unknown for a fully constructed and furnished building. The purpose of this research was therefore to understand the global response of the fully constructed three-storey post tensioned timber building. Serviceability seismic actions were specifically investigated to analyse the response. System Identification (SI) techniques, Finite Element (FE) modelling, FE model updating and time history analysis were used to identify natural frequencies, mode shapes and expected response. This work provides a validation of the FE model assumptions, indication of the real structure performance and opportunities for better design assumptions for the elastic performance.

1.1 Building Description

The NMIT Arts building resists horizontal loads using post-tensioned Laminated Veneer Lumber (LVL) shear walls based on EXPAN technology. Universal Flexural Plates (UFPs) couple two shear walls together and act as energy dissipating devices. In the event of a large earthquake, the shear walls are designed to rock with the post-tensioning providing excellent re-centring capability. Yielding of the UFPs absorbs seismic energy providing localised damage to minimise repairs after a major event (Iqbal et al. 2007). When rocking is induced, the fundamental period of the building increases, this in turn decreases the seismic forces. The NMIT building uses four coupled shear walls, two in each orthogonal direction.

LVL structural frames with non-moment resisting connections provide the support for vertical loading. The lightweight timber floor system uses proprietary "Potius" panels to span between the main frames. On top of the Potius panels is a 75 mm thick non-composite concrete flooring slab (Omenzetter et al. 2011).

1.2 Monitoring system

The building has been extensively instrumented to measure its various long term as well as dynamic responses. The instrumentation relevant to this research is the ten tri-axial accelerometers, as shown in Figure 1. These are made up of three on the ground and second floors, two on the first floor, and one on the top floor, as well as a further external free-field sensor to record ground motion. These sensors are aligned to the axes of the building, which run almost due east-west and north-south.

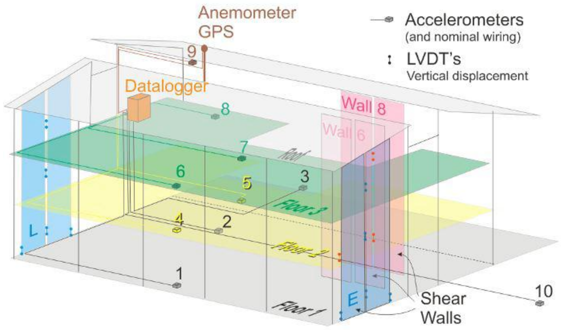


Figure 1 - Layout of sensors in NMIT Arts Building (numbers 1-10 indicate accelerometers).

2 FINITE ELEMENT MODEL

A FE model was initially provided by previous research (Worth 2011). After checking for consistency with the construction drawings it was found that amendments to this model would be necessary. Due to the nature of the amendments a new model was constructed in FEMtools, using the previous model as a reference. The aim of the new model was to reduce the number of nodes and elements within the model to increase the ease of model updating whilst still maintaining accurate results.

2.1 Important model information

To reduce the number of elements in the model, a shell element was used to represent the total roofing system. A shell element was chosen to model the roof because of previous testing suggesting semi-rigid behaviour by roofing diaphragms in timber structures (Phillips et al. 1993). Initial sensitivity analyses indicated that only in-plane stiffness and the mass of the roofing area was critical to the overall global response. In order to keep the in-plane stiffness the same, the original complex model of the roofing system was pushed with a 100N force at one end whilst fixing the other end. The deflection was recorded and an equivalent shell thickness was then calculated. This was performed by solving the below equations (1) and (2) iteratively using Microsoft Excel's solver add-in:

$$\Delta_s = \frac{VH}{GBt} \tag{1}$$

where V = horizontal force; H = length of roof; G = shear modulus of timber; B = breadth of roof; and t = thickness of diaphragm.

$$\Delta_b = \frac{VH^3}{3EI} \tag{2}$$

Where: V = horizontal force; H = length of roof; E = Young's modulus; I = second moment of area.

Stairwell was omitted from the FE model. Because it has roller connections at the foundation and pin connections at each floor level, it is likely that the stairwell would only add a small amount of mass and stiffness to the floor levels. Hence, this was a quick and effective way of removing several nodes from the model with little effect on the global response of the model.

The stiffness of the UFP coupler was another critical amendment. The model of the shear was system is based on recommendations made by Newcombe (Worth 2011). It utilises a frame member with equivalent section properties of the shear wall, coupled with vertical spring elements between the shear walls. The axial stretching of this spring was calculated based on the stiffness of two horizontal beam elements with fixed ends inducing shear like deformation.

Extra non-structural mass of 50 kg/m^3 was added to the concrete flooring. This was to allow for extra weight that would have been added by class room equipment. The beam column connections were modelled using two node springs. By altering these values manually and observing the effects it had on the model, it was found that the connections, at the low amplitude of excitation they were subjected to, must be effectively fixed.

2.2 Amendment affects

After the equivalent roofing shell element was in place, it was observed that its effect on the first natural frequency was very small, hence further agreeing that an equivalent diaphragm is an effective way of modelling a roofing structure. Even though the UFP stiffness was found to be not sensitive in updating, it was still important to have the initial value correct. Altering this spring constant by 1×10^{12} N/m would vary the first fundamental frequency by up to 1 Hz. Adding the extra weight did not have a significant effect: adding 50 kg/m^2 to the entire flooring area reduced the first natural frequency only by 0.2 Hz. The changes introduced into the model had reduced the first natural frequency from approximately 5 Hz to 3.48 Hz. This was very close to the first natural frequency of 3.52 Hz found by previous forced vibration testing (Worth et al. 2012).

3 SYSTEM IDENTIFICATION METHODOLOGY

3.1 Data records

Ten data sets were obtained for earthquakes over M5.0 on the Richter scale, but with epicentre far away from the structure. The acceleration trigger level for the data sets analysed in this research was 4.0 mg. Many of the records generated were from the February 2011 Christchurch earthquake aftershocks as well as the main shock.

3.2 Identification techniques

A system identification toolbox (SIT) developed at the University of Auckland (Beskhyroun 2011) was used for identification of modal parameters of the structure without repetitive, complex mathematic manipulation by hand, allowing time saving and removal of calculation errors.

The SI techniques used combined frequency and time domain approaches. In the frequency domain, Peak Picking (PP) (Peeters and De Roeck 2001) and Frequency Domain Decomposition (FDD) (Brincker et al. 2001) were used. In the time domain, Stochastic Subspace Identification (SSI) (Van Overschee and De Moor 1996) was used. Two variants of SSI featured in the SIT, one using the average of stable poles found in a user-defined band centred around frequencies found by the FDD method, the other averaging poles across an arbitrary frequency band defined again by the user. These are referred to as 'SSI' and 'SSI2', respectively. For this research the band used for SSI was 1.0 Hz and for SSI2, 0.1 Hz.

For a building of this height (13 m), it was expected that the majority of the responses would be in the range 0 - 10 Hz. Therefore the data was filtered in the frequency domain to remove responses above 20 Hz, using a cubic function curve rooted at the calculated spectrum value for 20 Hz. This filtering was completed on raw data before analysis so was applicable to all methods of system identification.

4 SYSTEM IDENTIFICATION RESULTS

4.1 Natural frequencies

Six modes were consistently identified, as shown in Table 1. Preliminary testing resulted in at least one mode being found in each of the transverse, longitudinal and torsional directions. Later collation of the analysis indicates mode {2} is a torsional mode rather than a longitudinal mode as initially expected, components of both torsional and lateral responses are possible due to prediction of both torsional and longitudinal modes by the FE model and FVT at around 4 to 4.3 Hz.

The majority of modes identified were transverse direction dominated. This is due to this being the shorter, less stiff, direction of the rectangular building, and due to the analysis being conducted on earthquake records alone, which may or may not excite all modes of the building. Torsional modes {2} and {4} on the other hand, were much more difficult to consistently identify. Only SSI2 was able to identify mode {2}, while SSI was not able to identify mode {4}.

Table 1 shows reasonable consistency between the different techniques in values of natural frequencies. The frequency values in Table 1 are averaged over the 10 data sets with outliers excluded, based on correlating mode shapes. An average frequency encompassing all techniques for each mode is also displayed in the first column of Table 3.

Table 1 - Natural frequencies, damping ratios and mode types for modes identified by each method.

Mode	Natural Frequency (Hz)				Type	Damping Ratio (%)	
	PP	FDD	SSI	SSI2		SSI	SSI2
{1}	3.13	3.13	3.41	3.26	Transverse	3.7	3.4
{2}	-	-	-	4.28	Torsional	-	3.5
{3}	5.47	5.47	5.81	5.81	Transverse	4.1	4.9
{4}	7.81	7.81	-	7.79	Torsional	-	3.6
{5}	8.59	8.59	8.73	8.46	Transverse	5.2	5.2
{6}	14.71	14.50	14.76	14.05	Transverse	4.6	3.6

4.2 Damping ratios

The damping ratio refers to the dimensionless measure of the decay of vibrations in a structure after excitation. Better knowledge of damping ratios would allow a better prediction of the building response. SSI was used to calculate damping ratios due to limitations in accuracy of predictions using frequency domain methods (Zhang and Cho 2009).

Table 1 also displays damping ratios for each mode found using the SSI and SSI2 techniques. These are average values over the 10 data sets with outliers excluded. These are generally quite close to each other, with the greatest difference being around one percentage point for mode {6}. Once again an average between the techniques is shown in Table 3. The damping ratios are fairly similar over all the modes, with a spread in average values from 3.5% for mode {1} to 5.2% for mode {5}.

4.3 Mode shapes

Identified mode shapes are displayed in Figure 2. 2D plots of the response in each axis of the building gave a prediction of the type of mode for each frequency. Modes were most strongly identified in the transverse direction, both in terms of transverse direction dominated modes, and the transverse component of other mode shapes. Mode {2} was found by a single technique only so is less reliable. Selection of final mode shapes was made by comparative plotting rather than use of MAC values due to having up to 10 similar shapes and the complexities posed by interpreting a matrix of MAC values of this size. This meant an average was the final outcome rather than a selection of one result as the representative shape. This would also act to smooth any small variations.

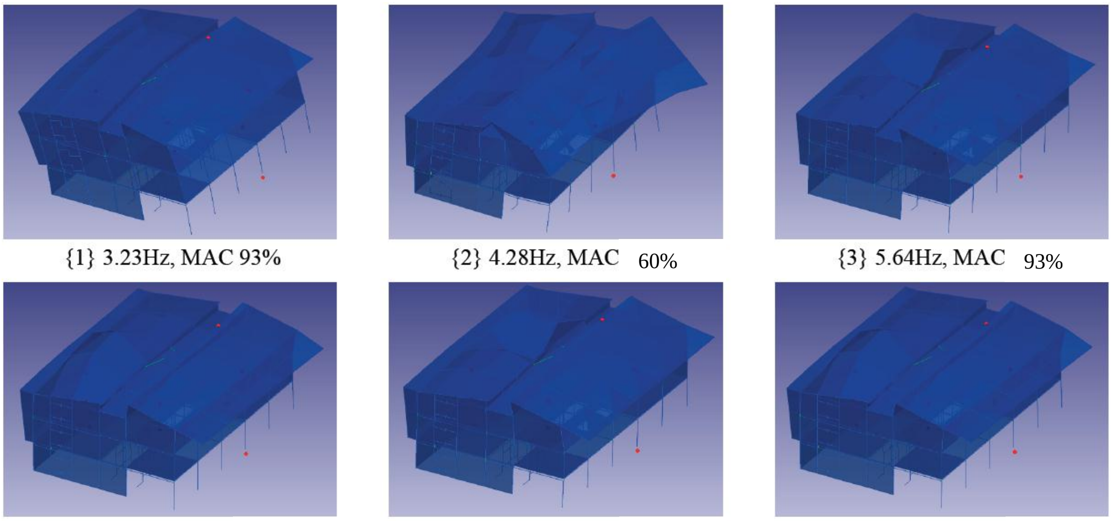
There is a limitation on the interpretation of mode shapes due to the number and positioning of sensors in the NMIT building. A total of only 9 sensors was used throughout the three-storey building , with a footprint of over 300 m^2. Furthermore, the sensors were not located on the corners of the building.

5 MODEL UPDATING

Model updating was performed to reduce the differences between the estimated natural frequencies of the FE model and the natural frequencies identified by the testing results. Model updating was completed with the system identification results outlined in section four of this paper.

5.1 Model correlation

The experimental mode shapes and FE model mode shapes were paired in FEMtools. This was completed automatically in FEMtools based on maximising the overall Modal Assurance Criterion (MAC) values. MAC values are simply a measure of how well an analytical mode shape fits to an experimental mode shape. A MAC value over 80% was considered a match and a MAC value over 90% was considered a very good match.



{1} 3.23Hz, MAC 93% {2} 4.28Hz, MAC 60% {3} 5.64Hz, MAC 93%

{4} 7.81Hz, MAC 41% {5} 8.60Hz, MAC 50% {6} 14.66Hz, MAC 90%

Figure 2 - Identified mode shapes.

To compute the MAC value in FEMtools equation (3) is used (Dynamic Design Solutions 2008):

$$MAC = \frac{\left|\left(\{\psi_a\}^t\{\psi_e\}\right)\right|^2}{\left(\{\psi_a\}^t\{\psi_a\}\right)\left(\{\psi_e\}^t\{\psi_e\}\right)} \times 100\% \tag{3}$$

where: ψ_a = analytical mode shape; ψ_a = experimental mode shape.

The seismic monitoring data paired fairly well with the FE model, however, only two mode shapes and frequencies were reliably paired. These were the pairs with experimental mode shapes {1} and {3}. Experimental mode shape {6} also had a very good pairing, however, the frequency was too far from the analytical frequency to make this a viable pairing (Table 3).

Table 3 - Mode Shape Pairing

#	FEA Mode	Frequency (Hz)	Experimental Mode	Frequency (Hz)	Diff. (%)	MAC (%)
1	1	3.48	1	3.23	7.72	92.8
2	7	6.96	4	7.81	-10.78	40.9
3	8	7.36	3	5.64	30.57	93.4
4	9	7.4	5	8.6	-13.93	50.1
5	14	8.14	2	4.28	90.33	59.7
6	15	8.24	6	14.66	-43.8	90.1

5.2 Sensitivity

Eleven parameters were chosen for the sensitivity analysis. The differential method was used to calculate the sensitivities of natural frequencies to structural parameters. Normalised relative sensitivities were used so that comparison was possible between the different types of parameters. A normalised sensitivity represents the percentage change of a response for a 1% change in the parameter value (Dynamic Design Solutions 2008). Six parameters were found to give large sensitivity, these were the: Young's modulus of concrete, LVL and cladding, as well as the non-structural floor mass, and the Iz value of the two main beams. However, because there were only two experimental mode shapes that could be used for updating, only four responses (two natural frequencies and two MAC values) could be used for updating. In order to have confidence with the updating results it was decided to use more responses than the number of parameters updated to when performing the model updating. Hence, three parameters would need to be eliminated.

Firstly, initial attempts were made to update the non-structural mass on the flooring. This was to assess if it were possible for an increase in mass to cause the difference in natural frequencies. It was found however, that this was not plausible thus, the mass was ruled out as an updating parameter. Lastly, the Iz values were ruled out. This is because when updating to the E and the Iz at the same time, an increase in the Iz value and a decrease in the E value can effectively cause no change in the natural frequency of the model. In such a case, rather than one single solution, a range of solutions are possible. Because the E of LVL was spread evenly throughout the building is was chosen to discard the Iz values.

5.3 Updating in FEMtools

Iterative sensitivity-based updating was performed on both sets of experimental data. The error function selected was the weighted absolute relative difference (CCABS) between resonance frequencies. The CCABS is calculated in FEMtools based on equation (4) (Dynamic Design Solutions 2008). Automatic updating was set to stop when: the CCABS error was below 1%; the change in the CCABS was less than 0.1%; or when the number of iterations reached 20. The initial CCABS value was equal 19.1%.

$$CCABS = \frac{1}{N}\sum_{i=1}^{N} C_{Ri}\frac{|\Delta f_i|}{f_i} \qquad (4)$$

where: N = total number modes selected; i = i^{th} mode; Δf_i = FE model frequency for mode i; Δf_i = difference between FE and experimental frequency for mode i; C_{Ri} = expected relative error on the response value.

Several iterations were performed by varying the initial conditions of each of the parameters by ±10%. The first set of updating runs varied those parameters around the originally set values. This yielded very stable results however the results were effectively rendered meaningless due to very large changes in parameter values. A second set of updating was based on the initial conditions form an earlier work by increasing the E of LVL to 16.1 GPa and decreasing the E of the cladding to 0.11 GPa. The results from this set were more meaningful, however, quite unstable. The median results from the second set of updating were then used as the initial conditions, producing very stable results.

Table 4 - Updated parameters

Parameter	Mean (GPa)	Std (GPa)	Coeff. of variation (%)	Change (%)
E LVL	12.8	0.067	3	16
E Concrete	4.0	0.167	22	-87
E Cladding	$0.12x10^{-6}$	$1.94\ x10^{-9}$	8	-100

The E of LVL increased from the design value of 11 GPa to 12.8 GPa (Table 4). This was expected because the design value is the lower 5^{th} percentile characteristic stiffness. Internal partitioning was also not modelled which would increase the stiffness of the building. The E of concrete however decreased significantly from 30 GPa to 4 GPa. This is of course goes against what was expected. A possible explanation for this is perhaps the way the slab is connected to the frame means the stiffness of the diaphragm is not being used. Future work into this is therefore required. Finally, the E of cladding also significantly decreased. This is expected to be representative of how the cladding is fixed to the building and that there are cut outs in the cladding, e.g. for windows.

5.4 Updated model correlation

The updated model improved the CCABS value and the mean MAC value also increased. The MAC mean improved from 72% to 78% and the CCABS from 19% to 2%. From table 5 it is easy to see that the difference between the responses selected for updating (mode {1} and {3}) improved considerably however, the MAC values for these two pairings have decreased slightly.

Table 5 - Updated Mode Shape Pairs

	FE Mode	Frequency (Hz)	Experimental Mode	Frequency (Hz)	Diff. (%)	MAC (%)
1	1	3.18	1	3.23	-1.72	90.8
2	6	5.75	3	5.64	1.95	85.5
3	7	5.85	6	14.66	-60.09	73
4	14	6.93	4	7.81	-11.17	71.4
5	15	6.96	2	4.28	62.58	59.6
6	20	7.13	5	8.6	-17.09	88.7

6 TIME HISTORY ANALYSIS

A linear time history analysis was lastly completed to estimate the maximum deflection that could be expected in a serviceability seismic event. The time history analysis was completed using the numerical Newmark (β=1/4) method in Excel. A simple method was adopted. The approach was to perform a time history analysis assuming the building was a single degree of freedom system. It was also assumed that the first mode shape would contribute to the majority of the response. Hence, only the response of the first natural period was assessed. Previous research (Oyarzo-Vera et al. 2011) was used to select appropriate serviceability seismic events. Seven scaled earthquake records were selected assuming a seismic zone NF. The scaling system was based on factors derived from NZS 1170:2005. It should be noted that the scaling method is valid for structures with natural periods greater than 0.4s. The first fundamental frequency of the NMIT building is 0.31s and hence it had to also be assumed that this had negligible effect on the earthquake scaling. It is hoped in the future to complete this area of the research in more detail.

The time history analysis has estimated that the NMIT building would indeed perform very well in a serviceability seismic event. The maximum scaled deflection was found to be approximately 8mm.

7 CONCLUSIONS

In order to understand the response of the innovative, multi-storey building in a serviceability level seismic event, SI using dynamic monitoring data, FE modelling, model updating and time history analysis was performed.

Consistency of modal parameters obtained experimentally was variable. The majority of the modes identified were in the range 3-10 Hz, with a fundamental frequency of 3.23 Hz and average damping ratio 3.5%. Modes {1}, {3} and {5} are considered reliable, the fundamental mode in particular. Mode shapes were taken as an average from all data sets and techniques excluding outliers, and are considered to be accurate for at least modes {1}, {3} and {5}.

There was substantial difficulty identifying modes other than those dominated by the transverse direction response. This is likely due to the orientation of the building and the excitation being by earthquakes which may not excite all modes.

Two frequencies and mode shapes from the system identification were used to update the FEMtools model to. The number of parameters being updated to was limited to three. The parameters were the Young's modulus of LVL, concrete and the cladding.

Updating found the E of LVL increased by 16% to 12.8 GPa, the E of concrete decreased by 87% to 4 GPa, and the E of cladding essentially decreased by 100% to 121 Pa. The decrease in E of concrete and cladding could be due to the way these elements are fixed to the building. Further investigation into this is required. The updated parameters increased the overall mean MAC value from 71% to 78% and decreased the CCABS error from 19% to 2%.

Finally, a simple time history analysis was completed. This was to gauge how well the NMIT building would perform in a serviceability seismic event. It has been estimated that the maximum deflection of the building will be 8mm. It is expected that the building will therefore perform very well.

8 ACKNOWLEDGEMENTS

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