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MEASURED RESPONSE OF INSTRUMENTED BUILDINGS DURING THE 2013 COOK STRAIT EARTHQUAKE SEQUENCE

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ABSTRACT

With the recent high level of earthquake activity throughout New Zealand there is growing awareness of the need for quick and reliable determination of whether buildings are safe. In parallel, on-going advances in sensors and computing technology have resulted in the potential for new and innovative sensing systems which could change the way that civil infrastructure is monitored, controlled and maintained.

Following the 21 July 2013, M_W 6.5 Cook Strait earthquakes, seven buildings in the Wellington region were instrumented with low-cost accelerometers to record building response data sets during aftershock excitations. A summary of the data analyses and insightful information obtained through processing and interpretation of the raw data is presented. Key challenges and considerations of installing a permanent structural monitoring system into buildings in New Zealand are discussed. The goal was to relate building performance indicators to decision making processes regarding the safety and resilience of structures post-earthquake. The information obtained was sufficiently reliable and valuable to the decision making process and New Zealand can expect more permanently instrumented buildings in the future.

INTRODUCTION

Civil infrastructure such as bridges, buildings, pipelines and dams are a central component necessary for a functional society, and play a key role in providing safety and security. Infrastructure is the largest investment of a country, and therefore the safety, resilience and mitigation of seismic risk is important.

The evaluation of the real-time health and performance of structures in the built environment is an area of increasing interest to engineers and building owners, particularly following the 2010-2011 Canterbury earthquake sequence and the more recent July 2013 Cook Strait and January 2014 Eketahuna earthquake sequences [1-3].

Following the Canterbury earthquakes, a panel of experts from the International Business Machines Corporation (IBM) identified key recommendations and opportunities that should be integrated into the Christchurch city rebuild. One of the eight recommendations was to integrate digital technologies into infrastructure to make the city more resilient and to position the city as a world leading smart city [4].

The rapid advancements in sensors and sensor technology over the past decade have meant that installing a monitoring system into buildings is becoming an increasingly viable option [5, 6]. Installing sensors into buildings allows data to be collected and interpreted to help understand how structures in the built environment perform. Through the analysis and interpretation of the recorded data, a building monitoring system has the potential to provide insightful information to building owners, occupants and engineers to assist with important post-earthquake decisions. The recorded data can also be used to verify and update the design guidelines for detailed seismic assessment of existing buildings. The results obtained from a building monitoring system could be used in parallel with the current practice of observational assessments of infrastructure.

This paper provides a summary of the challenges and practical considerations involved in the instrumentation of buildings in New Zealand. The instrumentation of seven buildings in the Wellington Central Business District (CBD) during the Cook Strait earthquake sequence is then discussed and an overview of the types of insightful information that can be obtained from building monitoring systems is presented.

INSTRUMENTATION OF BUILDINGS

Over the last decade buildings have increased in size and complexity, resulting in an increased number of necessary assumptions being introduced in the computer modelling and design stages of a structure. Therefore, the actual performance of a building may not replicate the modelled response.

Some of the factors that affect the structural performance of a building are modal characteristics, type of construction materials and regularity in plan and in elevation, as shown in Figure 1. Additional factors include soil conditions, distance to the earthquake source, magnitude of earthquake, and the presence of adjacent buildings. The response and performance of a structure during an earthquake is governed by a unique combination of these factors. The ability to monitor and estimate these factors in the built environment is an area of particular interest to engineers, especially at this time following the recent level of earthquake activity.

Standard practice for determining the integrity of a structure following an earthquake involves inspections where highly qualified personnel must visually inspect the building. It is a difficult task to determine the actual seismic performance of a building due to variations that arose during construction, the

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lack of structural drawings for older buildings and the unknown quality and type of materials used. Structural damage can occur within a structure and often be hidden from the human eye, such as the presence of internal cracks and bond failure in concrete columns, which consequently go unseen. Traditional visual inspections are often expensive, time consuming, and subjective in nature.

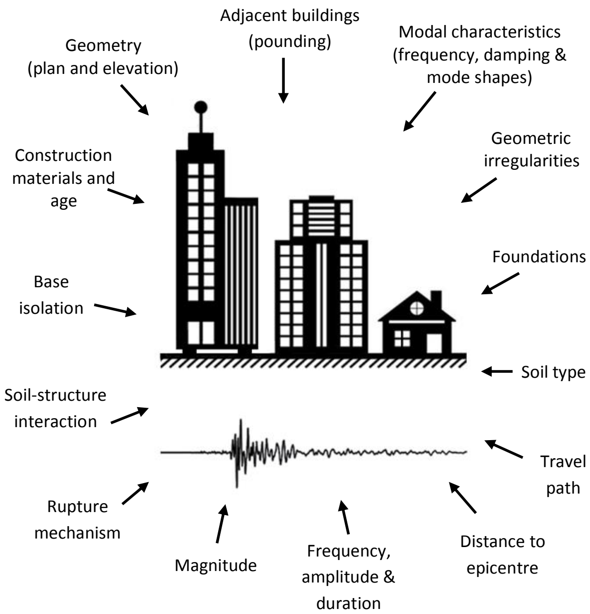


Figure 1: Some factors that affect the performance of a building during an earthquake.

Recorded data from a monitoring system can be used to verify and update the design guidelines for detailed seismic assessment of existing buildings. An ideal building monitoring system should record, process and analyse building performance in close to real-time, both before and after an event occurs. The data and results obtained from accelerometers could be used in parallel with the current practice of observational assessments, which are used to subjectively estimate the integrity of structures following an earthquake. There are currently no laws or guidelines in New Zealand that require seismic devices to be installed into buildings. By comparison, in the United States the Uniform Building Code (UBC-1997 and prior editions including the 2012 International Building Code [7]) recommend that a minimum of three tri-axial accelerometers are installed into buildings located in seismic zones that, a) exceed ten storeys in height, or b) exceed six storeys in height with an aggregate floor area greater than or equal to 60,000 square feet [8].

The GeoNet Building Instrumentation Programme was initiated by GNS Science in 2007 with funding from the New Zealand Earthquake Commission (EQC). The aim of the programme is to gain insights into the earthquake performance of buildings in New Zealand through the installation of seismic monitoring devices into 30 representative buildings [9]. It adopts a permanent instalment of seismic devices and utilises a GPS unit within the sensor network to facilitate time synchronisation between sensing nodes. Raw data collected from the instrumented buildings is freely available to scientists and engineers for research purposes. A research project at the University of Canterbury used the recorded data to conduct a preliminary investigation into the dynamic response of a selection of the GeoNet instrumented buildings during the 2013 Seddon and Lake Grassmere earthquakes [10].

Current Building Monitoring Systems

Over the past decade there have been major advancements in sensors and sensing technology [11, 12]. The vast amount of hardware and technology currently available can be utilised to monitor and capture data on building performance. Systems have been developed to assist with the diagnosis of civil infrastructure, which is commonly undertaken on a case-by-case scenario, as each structure requires different sets of algorithms to measure parameters that are of most interest. Future research seeks to assist with the prognosis of structures, such as using the data to provide real-time damage assessments and safety analyses.

Traditional sensing networks require cabling to connect all sensor nodes to a central repository unit. The need for cables has both positive and negative implications. A main advantage of utilising cables is reliable time synchronisation, as the cable provides common time stamps to all devices. Disadvantages of installing a dense array of traditional tethered sensors includes the significant cost associated with cabling, and the likelihood of the system becoming dysfunctional due to cable degradation and damage [13]. The amount of cabling required varies depending on the size of the structure and the desired density of instrumentation.

One promising approach that eliminates the need for extensive cabling is wireless sensing technology. Current research investigates the challenges that exist with wireless sensing technology such as the reliability, transmission, power supply, local data processing and time synchronisation [13, 14]. Wireless monitoring systems offer the potential for a large number of sensing nodes to be installed into civil infrastructure.

The duration of monitoring is an important aspect to discuss, as this predominantly has a direct impact on the costs of installation. A building monitoring system can either be installed for short-term temporary monitoring or long-term permanent monitoring. A temporary monitoring system can be installed directly after an earthquake occurs, to capture information on building performance post-earthquake. A permanent monitoring system can continuously record and monitor the integrity and performance of the structure over the full lifespan. The advantage of a permanent monitoring system is the continuous recording of data prior to a natural disaster, which can be used to create baseline information to better estimate the extent of damage immediately after an event. Ambient and forced vibration tests can be used to determine certain parameters prior to and directly after any large event [15].

Parameters to Monitor

A monitoring system in a building can be treated like an aeroplane's black box. The raw data can be extracted following an event to help determine how the structure performed and more importantly to establish if and why the structure failed to meet performance objectives. As shown in Figure 1, there are many factors that affect the performance of buildings, and consequently there are many parameters that can be monitored. The Cook Strait earthquake sequence provided a unique opportunity to talk to building owners, engineers and occupants in the Wellington region to understand the type of information that is of most interest to monitor.

The monitoring of inter-storey drifts, displacements, and intensity of accelerations experienced at ground level, and throughout the building are of interest to engineers. During the Cook Strait aftershocks the authors found that building owners were generally concerned about the reduction in business downtime and whether the building is safe to be reoccupied.

Other parameters of interest to monitor are modal characteristics, which are used by engineers in design and analysis of civil infrastructure. An accurate estimation of modal parameters is important for estimating the magnitude of forces and deformations that a structure will be subjected to during an earthquake. The change in modal parameters can also sometimes provide an indication of structural damage [16]. Therefore accurate determination of natural periods, mode shapes, and modal damping is a necessary and important task in the design of civil engineering structures [17] and the definition of seismic design loads according to the NZS 1170:2004 standard [18].

Density of Instrumentation

The monitoring of a structure can be performed at two different levels, which are categorised at the local and global level. Monitoring at the global level is associated with understanding the behavioural performance of the whole building. Monitoring at the local level is associated with the behaviour of an individual element inside a building, such as a concrete beam, and the performance of the materials that constitute that specific element. A dense array of instrumentation is needed to capture local behaviour due to individual members needing a large number of sensors, whereas a smaller number of sensors are required to capture the global response of a building. The estimation of global performance is considered here.

The density of sensors installed into a building dictates the reliability of output that can be obtained to characterise the global performance of a structure. A matrix comparing the density of sensors with the information output is shown in Figure 2. As a general rule, the richness of insightful information obtained increases as the density of instrumentation increases, as does the cost of installation and data processing.

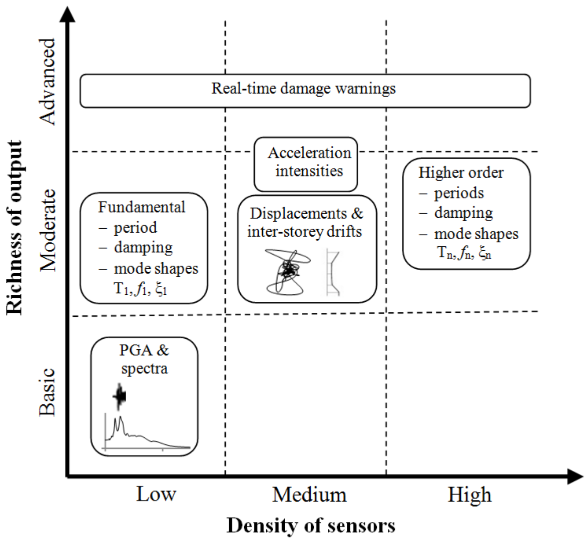


Figure 2: Density of sensors against richness of output matrix.

For the three categories of instrumentation, low, medium and high, it is recommended that a minimum of 3 sensing channels (i.e., 1 tri-axial accelerometer) are installed at basement or ground level to record the interaction response between the structure and the ground.

Low density instrumentation is useful for monitoring at the basic level and capturing input motions at the base of the structure, spectral accelerations and estimation of fundamental modal properties, but ineffective for analysing and producing advanced results, such as drifts and torsional effects. In addition to the tri-axial accelerometer installed at basement or ground level, a minimum of two tri-axial accelerometers are recommended for a low density system, with these sensors installed at mid-height and roof level, as shown in Figure 3a. The number of sensors required for low density instrumentation is independent of the height of the structure.

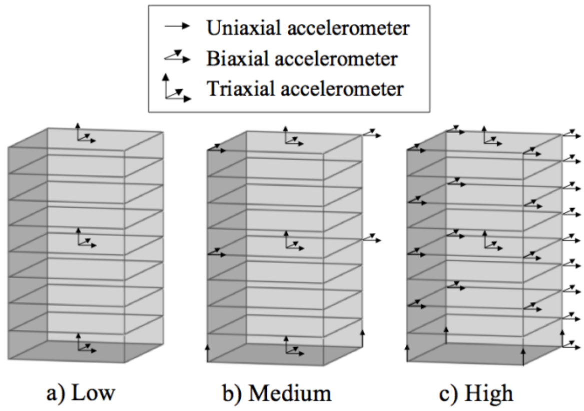


Figure 3: Example of the minimum recommended number and placement of sensors for the three densities of instrumentation options for an 8 storey building.

Medium density instrumentation aims to capture more advanced data than low density instrumentation for producing more insightful and reliable information such as displacements, inter-storey drifts and torsional effects. The number of sensors recommended for floors above ground is proportional to the height of the building. For buildings with 10 storeys or less, it is recommended that a minimum of 12 sensing channels are installed above ground. For buildings with 11 to 20 storeys, the recommended minimum number of channels installed above ground is 15. Ideally the sensors above ground are installed at evenly spaced floor levels and mounted at the outer perimeter of the structure, as shown in Figure 3b. These medium density instrumentation requirements satisfy the guidelines recommended by the Advanced National Seismic System (ANSS) of the United States Geological Survey (USGS) [19].

High density instrumentation involves sensors to be installed at most corners of every floor, at locations where building stiffness is expected to change, at locations where higher modes are expected to contribute the most, and to be distributed along walls and diaphragms that are of particular interest, as shown in Figure 3c. A larger number of sensors are required to ensure that the data is sufficient to perform meaningful model verifications, especially in the cases where significant rocking or torsion is expected.

INSTRUMENTATION PROCEDURE AND CONSIDERATIONS

A decision tree of the procedure for instrumenting buildings in New Zealand is presented in Figure 4, which was inspired by the authors' experiences in Wellington following the Cook Strait earthquakes and from the ongoing research conducted by GeoNet [20]. Discussions with building owners and managers in Wellington, following the Cook Strait earthquakes, revealed that the majority of parties had interest in the concept of a building monitoring system. The five key steps that are involved in the process of instrumenting a building are: discussions with owner/engineer; installation of system; recording of data; interpretation of data; and providing insightful information for decision making purposes. Key challenges and considerations of installing a permanent

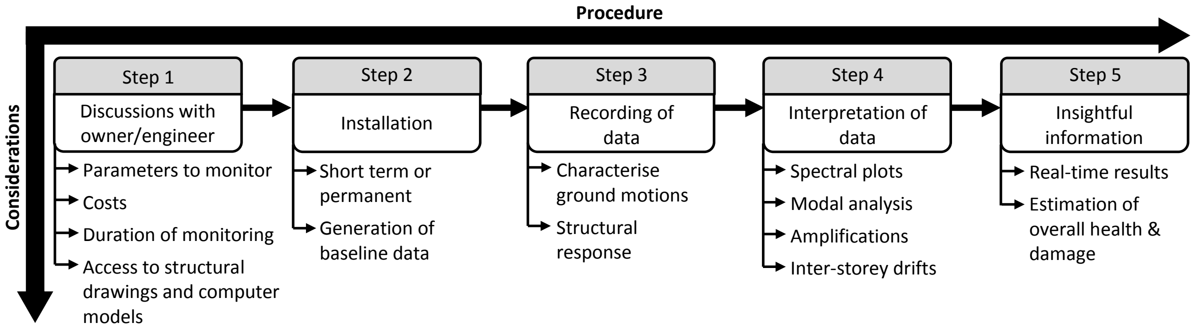


Figure 4: Decision tree of procedure and considerations for the instrumentation of buildings.

structural monitoring system into buildings in New Zealand are discussed in more detail to highlight the complex nature of this process. Initial discussions of the instrumentation process with an engineer, or expert in the field of seismic instrumentation, allows the parameters to monitor and feasibility of installation to be determined. Engineers can access the appropriate structural drawings and computer models. These tools are important inputs to the decisions on sensor placement. The desired density of sensors needs to be chosen as this dictates the installation cost and level of output that can be obtained, as discussed in conjunction with Figure 2.

Large amounts of raw data generated by the sensor array require interpretation to obtain useful information, which is often a challenging process. It is important to characterise an event so that building performance data collected from one event can be compared to data collected from other similar events. A description of commonly used techniques for characterisation of the ground motions and the structure are discussed, and are later referred to in the results presented in the "Instrumentation of Buildings in Wellington" section.

Characterisation of the Ground Motions

Time-history responses allow the peak ground acceleration (PGA), velocity and displacement at the site to be captured. Peak values are useful for comparing the relative intensities of ground motion events, but do not take into account the frequency content and duration of the event which has a strong influence on the amplification of vibrations throughout a structure. Response spectra plots include the frequency characteristics of a ground motion and are a convenient tool for quantifying the demands of an individual event on the capacity of a structure to resist an earthquake. Free-field accelerometers installed at ground level outside the building footprint capture the response spectra experienced at the site of the building for each ground motion event. Recorded response spectra can be compared to the structures design spectra, which takes into account the location and the site subsoil class for each building, as discussed in NZS 1170.5 [18]. The 5% damped elastic response spectra for six aftershock events, recorded at ground level of an instrumented building during the Cook Strait aftershock sequence is shown in Figure 5. In this case, the acceleration records were recorded within the building footprint, and would therefore be affected by soil-foundation-structure interaction effects.

Response spectra alone are insufficient to define all characteristics of ground motion events. Important damage-causing characteristics of ground motions include duration, amplitude and frequency. Once the amplitude and frequency contents of a signal are significant enough for nonlinear response to occur, the duration of the ground motion becomes a primary determinant of excessive damage [21].

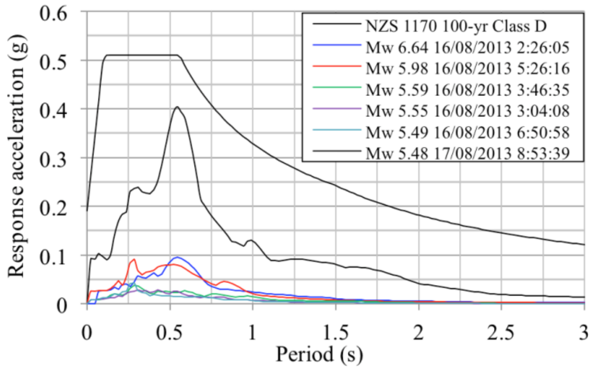


Figure 5: Elastic acceleration response spectra (5% damping) recordings for six ground motion events during the Cook Strait earthquakes.

The duration of separate ground motions can range from a few seconds up to several minutes, and this variation has a significant impact on cumulative damage. Additional intensity measures have been proposed to describe the damage potential of a ground motion [22]. Common engineering measures used to define the duration of an event include the Arias intensity, Housner intensity, root mean squared (RMS), acceleration and cumulative absolute velocity (CAV) [23, 24].

Characterisation of the Structure

There are a range of techniques and system identification methods available for processing and analysing acceleration data to provide accurate estimations on building performance. The methods investigated in the reported study for estimating modal parameters include three frequency domain based methods and one time domain based method. The frequency based methods are peak picking (PP) [25], frequency domain decomposition (FDD) [26] and enhanced frequency domain decomposition (EFDD) [26, 27]. The time domain method is the stochastic subspace identification (SSI) method, with two variations being used [28, 29]. These methods were identified using the System Identification Toolbox (SIT) program developed at the University of Auckland [30].

An example of the stochastic identification method for identifying translational modes of an instrumented building during the Cook Strait aftershock sequence is shown in Figure 6. The vertical lines of red dots (stability nodes) and the peaks in the spectral line graph represent the first seven identified frequency modes. The fundamental translational frequency was estimated to be 2.1 Hz (0.48 seconds) for this particular building and event.

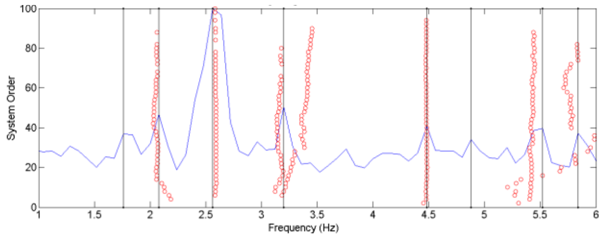


Figure 6: Stability nodes and power spectral density values of a ground motion record from the Cook Strait earthquake using SSI method.

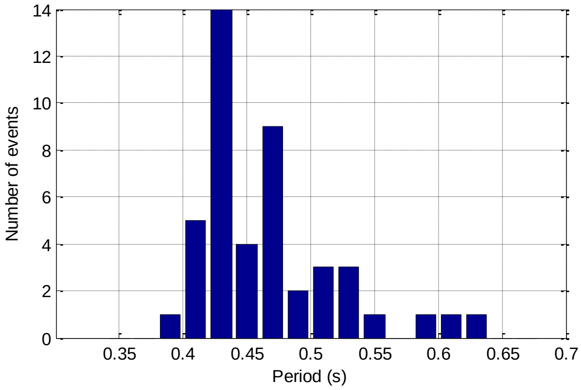


Figure 7: Distribution of estimated fundamental period from 45 consecutive ground motion events during the Cook Strait earthquakes.

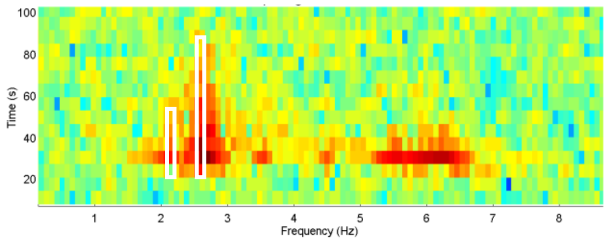


Figure 8: Spectrogram of a ground motion record during the Cook Strait earthquake sequence.

Multiple events need to be considered to gauge uncertainty levels of estimated building parameters. Figure 7 shows the fundamental period of a building estimated from 45 consecutive events during the Cook Strait aftershock sequence. The 45 events included in Figure 7 ranged in ground motion intensity from low ambient vibrations to 0.0128g, with an average period of 0.47 s and standard deviation of 0.06 s.

The change in the frequency content of a ground motion signal indicates that different modes were excited over time. A spectrogram reveals the frequency content of an aftershock signal as a function of time. Figure 8 reveals a spectrogram of a ground motion record from an instrumented building during the Cook Strait aftershock sequence. The two white boxes represent the first two modes that were identified for the building, and are highlighted by the areas of darker colours in the spectrogram. The vertical axis represents the time intervals in which different modes were identified during the ground motion record. The two white boxes highlighted in Figure 8 reveal that the first two modes were initially identified at the time of the largest PGA at approximately 25 seconds, and that the second mode was excited for a longer duration of time, as indicated by the taller white box.

The experimental measurement of inter-storey drifts and displacements are difficult due to the lack of a stationary reference frame up the height of the building to allow reliable estimations of drift measurements. High quality acceleration data can be analysed for drift estimations using numerical integration techniques [31]. Depending on the type and specifications of the sensing equipment, the data processing for drift estimations using this technique may require filtering, time synchronisation and baseline correction to the integrated data sets. For example, if the predominant periods of the structure are notably less than the maximum useable period of the sensors then baseline correction and filtering can remove the long period noise. Time synchronisation to the data sets is necessary if the individual sensors are not connected by cables which provides common time stamps to all devices. Methods for estimating additional building performance indicators, such as the presence of pounding, are discussed in the instrumentation of buildings in Wellington exercise that follows.

INSTRUMENTATION OF BUILDINGS IN WELLINGTON

The Cook Strait earthquake sequence began with the M_W 6.5 Seddon earthquake on 21 July 2013, which was located approximately 25 km Northeast of Seddon (41.60°S, 174.33°E) at a depth of 13 km [4]. During the period from 21 July to 16 August 2013 there were more than 2500 aftershocks recorded in the magnitude range (M_L) of 2.0 to 6.6. The locations of the earthquake epicentres were approximately 50 km from the Wellington CBD, as shown in Figure 9. The largest earthquake during this sequence was the M_W 6.6 Lake Grassmere earthquake that occurred on 16 August 2013 at a location about 20 km Southwest of the epicentre of the Seddon event (41.73°S, 174.15°E) at a depth of 8 km [4].

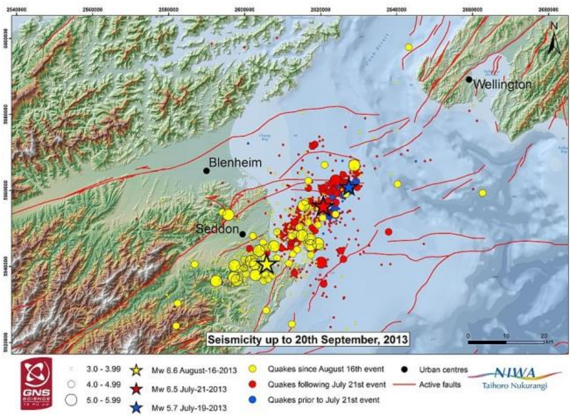


Figure 9: Locations of the Cook Strait aftershocks, courtesy of GNS Science [2].

Following the Cook Strait earthquake sequence the observed damage to infrastructure in the Wellington CBD included minor liquefaction damage (predominately in reclamation areas), varying levels of damage to buildings, and extensive slumping and ground cracking to pavements and roads. Refer to Holden *et al.* [28] and Van Dissen *et at*. [29] for further information on the ground motion characteristics and damage effects observed following the Lake Grassmere earthquake.

The Cook Strait earthquake sequence provided a unique opportunity to install seismic monitoring devices into buildings to record data during an aftershock sequence. Following the initial M_W 6.5 earthquake on 21 July 2013, seven buildings in the Wellington CBD were instrumented with low-cost accelerometers. The exercise was initiated within 48 hours as a reactive response to the first M_W 6.5

earthquake. The buildings that were instrumented varied in location, size, geometry, age and type of construction materials, which allowed data sets to be collected from a representative selection of the building stock of New Zealand.

Instrumented Buildings

In total seven buildings located in the Wellington CBD were instrumented during two different monitoring periods in the Cook Strait aftershock sequence, as shown in Table 1. Five buildings were continuously monitored from 21 July until 30 August 2013, and for confidentiality reasons are referred to as buildings ‘A’, ‘B’, ‘C’, ‘D’ and ‘E’. The details and results obtained for Building E are not included in this paper, as requested by the building owner/s. The last two buildings, the Saint Mary of the Angels Church and the Old Public Trust building, were instrumented continuously from 9 September until 23 October 2013 and are referred to as buildings ‘F’ and ‘G’, respectively.

Table 1: Identification and monitoring periods for the instrumented buildings in Wellington CBD.

Building ID		Monitoring Period	
ID	**Name**	**1**	**2**
A	---	✓	
B	---	✓	
C	---	✓	
D	---	✓	
E	---	✓	
F	Saint Mary of the Angels Church		✓
G	Old Public Trust Building		✓

Type of Sensors Used

The accelerometers used in this demonstration exercise were able to be mounted at any location within the building and each accelerometer had an individual power supply which meant that no central repository unit or wiring between the sensors was required. All sensing devices had sufficient on-board data storage to operate continuously at a sampling frequency of 40 Hz for approximately three weeks. The sensors were able to measure and record accelerations in the longitudinal, transverse and vertical planes of the instrumented buildings. Accuracy and limitations of these sensors have previously been tested and determined through quantitative analyses [32, 33].

Density of Sensors

Each of the instrumented buildings had a different number of accelerometers installed, which ranged from 6 to 32, based on the geometry, size and desired richness of output. At least one accelerometer was installed at ground level to record the foundation level response of the structure. The remaining sensors were placed at various levels and locations within the structure, specifically at the extreme boundaries to help capture the global response and dynamic characteristics of each building.

Buildings A to F were instrumented with a relatively low number of accelerometers and from Figure 2 can therefore be categorised as low-medium density instrumentation. These six buildings demonstrate the type of information that can be obtained from a simple and low cost building monitoring system. Building G had a medium-high density of instrumentation to investigate higher levels of output, as highlighted in Figure 2.

Recorded Events

In total, the accelerometers recorded approximately 540 separate aftershocks having a magnitude greater than M_L 3.0 during monitoring periods 1 and 2, as shown in Figure 10. The accelerometers proved to be very sensitive and recorded high quality building response data sets with good signal to noise ratios.

During monitoring period 1, the largest ground level acceleration recorded at an instrumented building was 0.175g in the horizontal direction during the M_W 6.6 Lake Grassmere earthquake on 16 August 2013 [34]. The ten largest ground motions that were recorded at the sites of the instrumented buildings during the complete monitoring period are shown in Figure 10. The moment magnitude (M_W), distance from the epicentre to Wellington CBD and depth of each aftershock are included in Table 2, as identified by GeoNet (www.geonet.org.nz). It can be noted that the ten largest ground motions all occured during monitoring period 1.

Building Response

For each of the instrumented buildings, only the parameters and results that are unique and of most interest to that particular structure are presented. The results aim to provide an overview of the types of insightful information that can be obtained from building monitoring systems of different density of instrumentation.

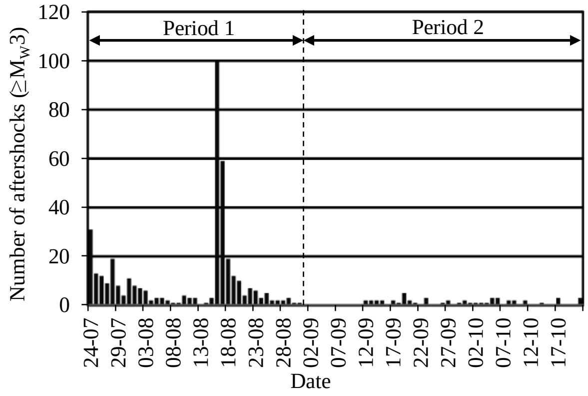


Figure 10: Number of aftershocks greater than M_L 3.0 recorded during the two monitoring periods.

Table 2: Details of the ten largest ground motions recorded during the Cook Strait aftershock sequence.

Epicentre magnitude (M_W)	Max Horizontal PGA (g)	Distance (and depth) to Wellington CBD (km)	Time (day/month -time)
6.64	0.17500	72.6 (8.2)	16/08-14:26
5.98	0.01834	61.0 (14.3)	16/08-17:26
5.59	0.02169	68.3 (19.4)	16/08-15:46
5.55	0.00971	79.5 (8.5)	16/08-18:04
5.49	0.00725	60.1 (19.6)	17/08-20:50
5.48	0.00971	81.6 (19.6)	16/08-18:53
5.07	0.00730	59.5 (5.1)	16/08-17:37
5.04	0.00874	75.4 (20.1)	18/08-04:02
5.02	0.00574	55.4 (8.5)	18/08-14:51

Buildings A and B

Buildings A and B both have regular geometry and a low density of sensors was installed. Because these buildings were instrumented during the same monitoring periods and the results obtained were similar, only Building A will be discussed in this section. A summary of the details of Building A are provided in Table 3. The nine accelerometers installed in Building A were mounted at the extreme perimeter of the structure and spaced across different floor heights, as shown in Figure 11.

The largest ground level excitation recorded at Building A was 0.0128g during a M_W 5.4 aftershock. A modal analysis of the data collected during the M_W 5.4 aftershock was conducted, with the fundamental period of the building estimated to be 0.61s in the transverse direction.

Table 3: Details of Building A.

No. of storeys	12
No. of accelerometers	9
NZS1170.5 Site subsoil class	C
Monitoring period (days)	28
Construction type	RC frame
Construction year	1970s

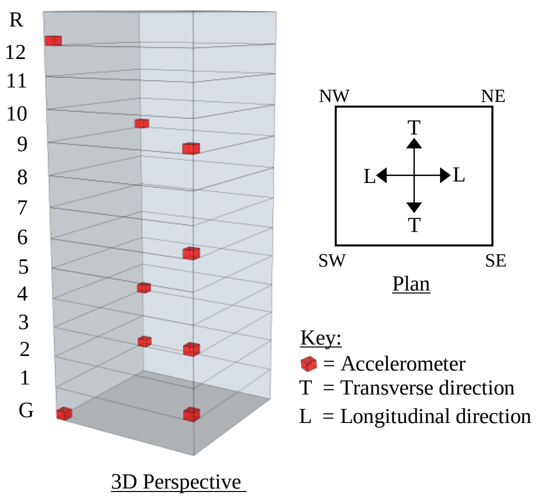
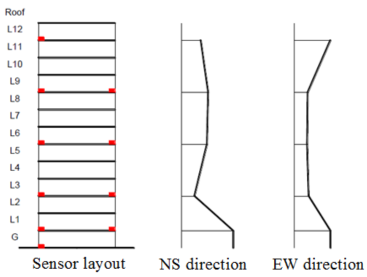


Figure 11: Layout and distribution of accelerometers in Building A.



Level	NS (mm)	EW (mm)
L12	0.28	0.53
L9	0.39	0.20
L6	0.37	0.19
L3	0.19	0.22
L1	0.75	0.53

Figure 12: Estimation of the maximum inter-storey drifts for Building A during a M_W 5.4 aftershock [31].

Inter-storey drifts were estimated through the double integration of the acceleration datasets, as discussed by Ma et al. [31]. The maximum average floor-to-floor inter-storey drifts for Building A were estimated at the centre of each instrumented floor, as shown in Figure 12. Interpolation of drifts at floors with no accelerometers installed was required due to the low density instrumentation. In order to get drifts without requiring interpolation it would be necessary to have instrumentation at every storey height. The low density monitoring system deployed in Building A was useful in estimating the fundamental periods of the structure along with interpolated mode shapes and inter-storey drifts during the M_W 5.4 aftershock.

Buildings C and D

Buildings C and D were both concrete multi-storey structures, eight and seven floors high, respectively. The buildings are adjacent to one another, with a clear seismic gap of approximately 100 mm. Seven accelerometers were installed into each building: one at ground/basement level and two sensors at each of the extreme corners on floors 1, 3 and 6. Refer to Figure 13 for more details on Buildings C and D.

The accelerometers at ground/basement level in Buildings C and D recorded PGA values in excess of 0.127g during the M_W 6.6 Lake Grassmere earthquake. Elastic response spectra were produced for both buildings during the M_W 6.6 event to help determine the characteristics of the earthquake and to estimate the demand on the two structures. The response spectra recorded at ground level for Building C in the transverse, longitudinal and vertical directions are shown in Figure 14.

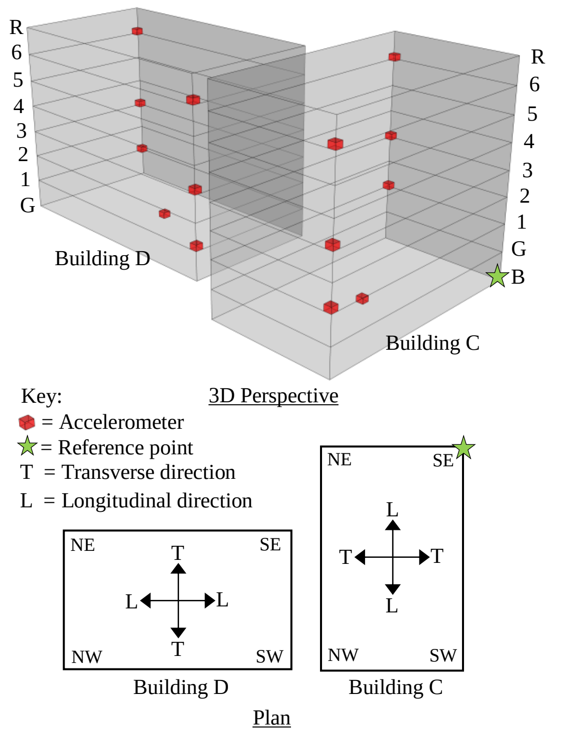


Figure 13: Layout and distribution of accelerometers in Buildings C and D.

The horizontal response accelerations during the M_W 6.6 event remained below the NZS 1170.5 100 year response spectra for a building in Wellington on soil type D. The accelerometers installed at levels 3 and 6 were able to verify the amplification in acceleration response up the height of the building. The fundamental periods of Buildings C and D were estimated from a modal analysis to be 0.57s and 0.83s, in the transverse direction, respectively [26].

The presence of pounding between Buildings C and D was investigated using the recorded acceleration data. The acceleration time-history responses in the transverse direction of Building C and longitudinal direction of Building D during the M_W 6.6 event are shown in Figures 15a and 15b. The estimated displacement time-history responses at level 6,

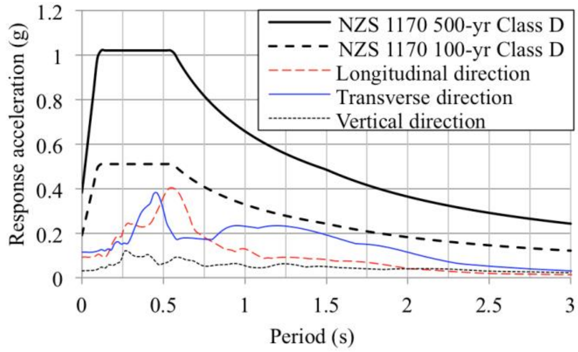
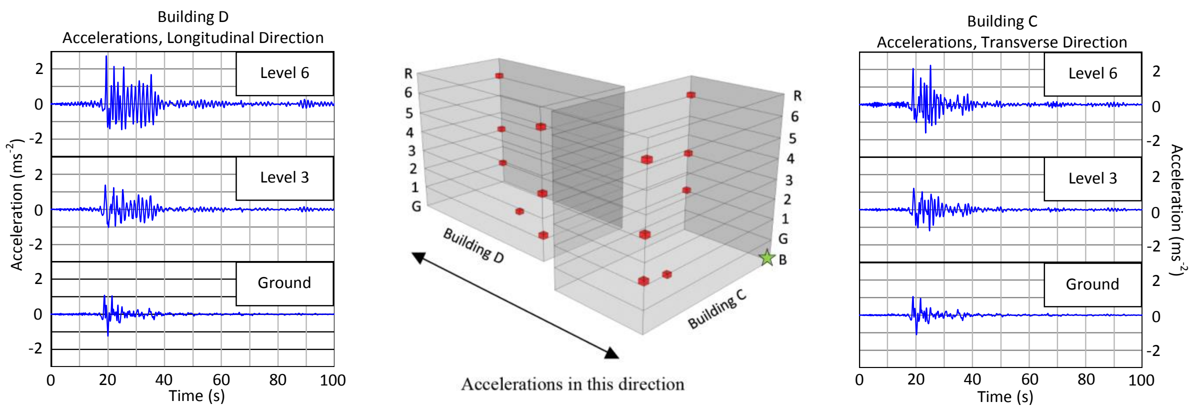


Figure 14: 5% elastic response spectra for Building C during the M_W 6.6 event.



A) Building D

B) Building C

Figure 15: Acceleration response of Buildings C & D in the pounding direction during the M_W 6.6 event. The accelerations at ground, Level 3 and Level 6 are shown for both buildings.

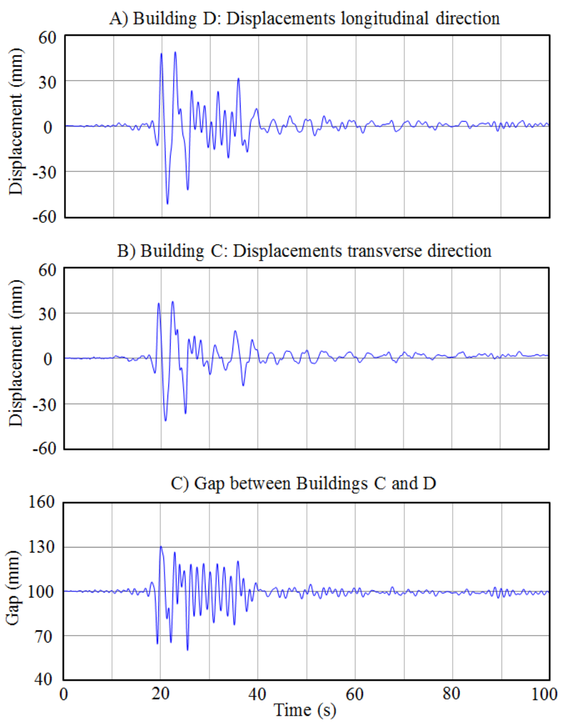


Figure 16: Estimated time history responses of the displacements at level 6 of Buildings C & D during the M_W 6.6 event, along with the relative gap between the two buildings.

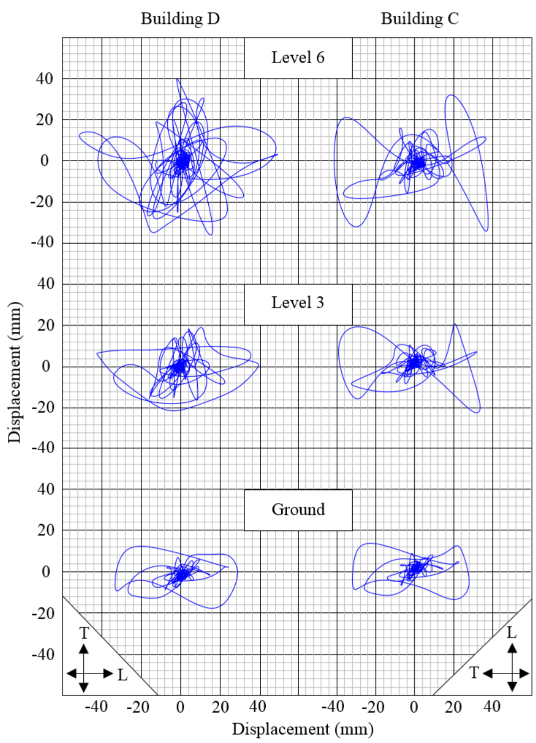


Figure 17: Estimated in-plan displacement trajectories for Buildings C & D during the M_W 6.6 event.

where maximum displacements were assumed, for Building C in the transverse direction and Building D in the longitudinal direction are shown in Figures 16a and 16b, respectively. The real-time opening and closing of the gap between the two buildings at level 6 is shown in Figure 16c. The gap was initially 100 mm and during the M_W 6.6 event closed to 60mm. Since the estimated gap did not reduce to zero no pounding between the two buildings occurred, this corresponded with observations.

Displacement data was estimated through the double integration of accelerations method to produce the displacement trajectories shown in Figure 17. The estimated displacement trajectories at ground level for both buildings was found to be near identical, as expected for two adjacent buildings. The displacement trajectories shown in the top region of Figure 17 reveal that the maximum displacements estimated at level 6 for Buildings C and D were 41 mm in the transverse direction and 51 mm in the longitudinal direction, respectively.

Building F (Saint Mary of the Angels Church)

The Saint Mary of the Angels Church was built in the early 1920s and is located in the heart of Wellington city. An image of the building is shown in Figure 18. During the Cook Strait earthquakes the Saint Mary of the Angels Church experienced low levels of structural damage.

Six accelerometers were installed in two groups of three, as shown in Figure 19. Each group of accelerometers was installed in the vertical plane, with one accelerometer on ground level, one at first floor height and the third mounted at roof level. One group of accelerometers was installed on the southern side of the portal frame near the middle of the church while the second group was installed in the southern tower at the church entrance. Details of Building F are provided in Table 4 and Figure 19.

Figure 18: Saint Mary of the Angels Church.

Table 4: Details of Building F.

No. of storeys	1-3
No. of accelerometers	6
NZS1170.5 Site subsoil class	C
Monitoring period (days)	28
Construction year	1920s

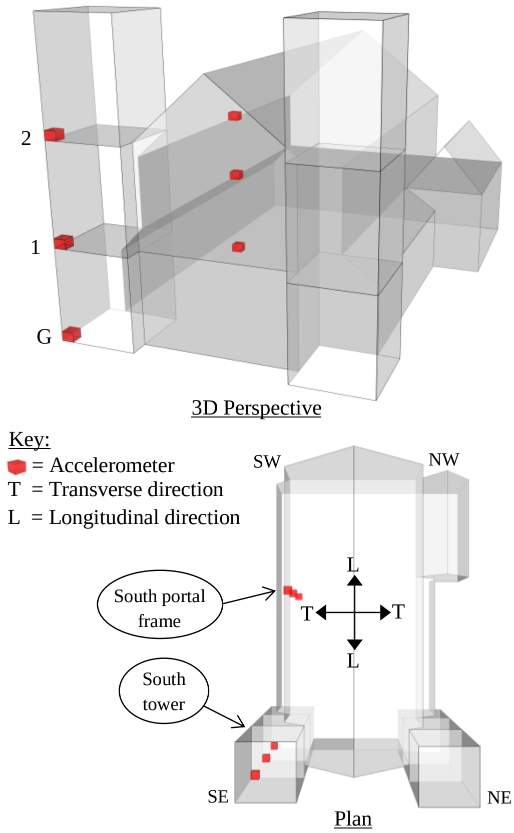


Figure 19: Layout and distribution of accelerometers in the Saint Mary of the Angels Church.

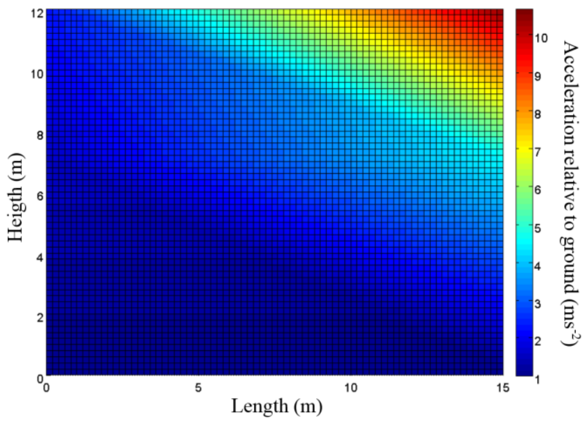


Figure 20: Out-of-plane acceleration profile of the southern wall in the Saint Mary of the Angels Church during the M_W 4.3 event.

The data collected from the accelerometers was used to determine the fundamental frequency values for the main body of the church and for the instrumented southern tower. The first modal frequency of the church was identified as 3.5 Hz in both the transverse and longitudinal directions.

The dissipation of energy in the southern wall diaphragm during the aftershock excitations was investigated by using the acceleration intensity distribution technique. The acceleration intensity distribution technique is useful as it is the integration of acceleration records over time, which is indicative of vibration energy. The measured out-of-plane accelerations relative to ground accelerations in the southern wall diaphragm during the M_W 4.3 event are shown in Figure 20 using interpolation of sensor data. The colours in Figure 20 indicate the level of accelerations relative to ground, where red represents a high level of acceleration and blue represents a low level of acceleration. The top right section of Figure 20 reveals that the out-of-plane accelerations in the southern wall diaphragm increased up the height of the wall, with accelerations approximately ten times larger than the recorded ground accelerations. This finding indicated that the top section of the wall at the western end responded in a flexible manner.

Building G (Old Public Trust building)

The Old Public Trust Building was erected in 1908 and is listed as a Category 1 heritage structure. The building is stone masonry construction and has an internal steel skeleton that supports the floors and facades. The building was strengthened in the 1920s and again in the 1980s. A view of the building from Stout Street is shown in Figure 21. A site investigation and installation of 33 accelerometers was undertaken on 9 September 2013. During the Cook Strait earthquakes, the Old Public Trust building experienced low/medium levels of structural damage. Large cracks appeared in the underside of the floor diaphragms, as shown in Figure 22.

Levels 3 and 4 received a greater number of accelerometers than the floors beneath, with 10 accelerometers installed on each floor. The layout of accelerometers and details of Building G are found in Table 5 and Figure 23.

Table 5: Details of Building G.

No. of storeys	5 + Basement
No. of accelerometers	33
NZS1170.5 Site subsoil class	C
Monitoring period (days)	28
Construction material	Stone masonry
Construction year	1908

Figure 21: Old Public Trust building in Wellington.

Figure 22: Underside of flooring system with location of cracking indicated by bold lines.

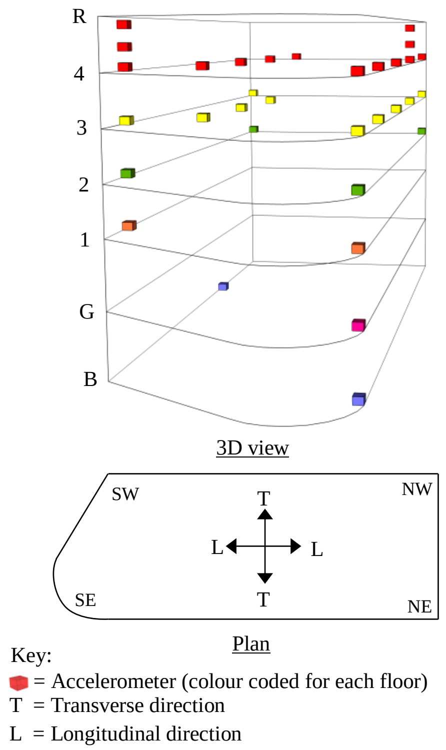


Figure 23: Layout and distribution of accelerometers in the Old Public Trust building (Building G).

Table 6: Estimated fundamental period of Building G from system identification in the transverse direction and using the empirical code equation found in NZS 1170.5.

	Period (s)	Damping (%)	NZS 1170.5 (s)
Mode 1	0.48	5.7	0.70
Mode 2	0.40	3.5	-
Mode 3	0.23	1.7	-

The number and magnitude of aftershocks during the second monitoring period were significantly less than during the first monitoring period, as shown in Figure 10. During the monitoring period for Building G, the largest recorded excitation at ground level was 0.0057 g during a M_W 4.67 event on 2 October 2013. These small intensity events recorded data with adequate signal to noise ratio to allow a detailed analysis to be conducted.

Due to the medium-high density of instrumentation in Building G, the mode shapes and the period and damping values for the first three modes were able to be estimated with a high level of confidence and accuracy. The period and damping values are shown in Table 6, along with the period estimation using the equation in section C4.1.2.2 of NZS 1170.5 Supp [35], which was found to be 0.70s and therefore a slight over-estimation. It must be noted that the predictive equations for period as given in NZS 1170.5 are not designed to represent the period of the structure at low levels of shaking. As the calculated period of a structure varies depending on the input intensity of shaking, it is difficult to determine the true period of Building G from the small intensity events. The research by Thomson et al. also found that the equation in NZS1170.5 often results in an over-estimation of building period, which could result in an underestimation of design forces [10].

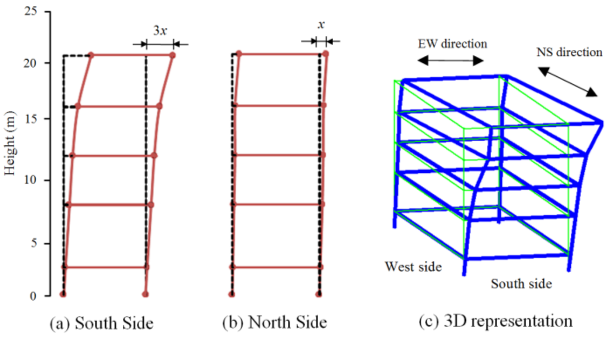


Figure 24: Normalised first translational mode shape at both ends of Building G in the EW direction.

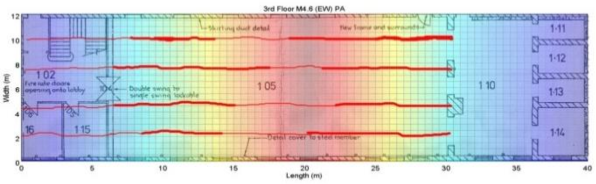


Figure 25: Acceleration profile of floor diaphragm in Building G which correlates to observed damage. Red lines represent floor cracking that developed during the Cook Strait earthquake sequence.

In Buildings A to F the interpolation of modal amplitudes was necessary to obtain values at all floor locations and at all four corner columns due to the low density of sensors. The data collected from Building G, which had a dense number of sensors at many corner locations and floors, were most effective in producing reliable first order mode shapes. The first translational mode shape was calculated in the transverse direction, due to the rectangular geometry of the building. Figures 24a and 24b reveal the normalised modal amplitudes of the first translational mode shape for Building G at the Southern and Northern ends of the building, respectively. It was found that the modal amplitudes at the Southern side were nearly three times larger than at the Northern side, which can be shown by the 3D representation of the first mode shape in Figure 24c. The difference in modal amplitudes at each end of the building is due to the relatively high stiffness at the Northern end from structural strengthening which occurred in the 1980’s.

From the data collected from Building G the acceleration profiles of wall and diaphragm elements could be produced. Figure 25 shows the in-plane acceleration profile of a damaged diaphragm. The colours indicate the level of accelerations, where red represents high and blue represents low levels of acceleration. The analysis allowed the actual response of an element within the building to be monitored to better understand how that element performed during extreme loading events. The increased accelerations near the centre of the diaphragm indicate that a flexural behaviour occurred, which correlates with the observed cracking patterns shown in Figure 25. In addition to monitoring the behaviour of diaphragms, the out-of-plane wall accelerations can be captured through a dense array of instrumentation which allows the true performance and actual forces experienced up the height of a building during any earthquake to be known.

CONCLUSIONS

The July 2013 Cook Strait aftershock sequence provided a unique opportunity to instrument seven existing buildings in the Wellington region with sensitive and low-cost accelerometers to record data on building response. The data collected from the seven instrumented buildings was of high quality and enabled insightful information to be estimated with a good level of confidence. Stable modal periods and mode shapes were defined for each instrumented building over the range of earthquake intensities, aligning with the lack of any significant damage during these events. Modal characteristics were also used to refine computational models at a number of the study buildings. The estimation of inter-storey displacements and acceleration profiles of wall and floor elements was able to help explain damage patterns observed following earthquakes, and the interaction, or lack thereof, between two adjacent buildings.

It was shown that the density of instrumentation influences the richness and complexity of information that can be obtained. Increased density improved understanding and reliability of the global dynamic response of the building and the response of elements within each building. The insightful information obtained proved to be valuable and sufficiently reliable. More permanently instrumented buildings in New Zealand can be expected in the future.

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