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DEVELOPMENT OF A MONITORING SYSTEM FOR DAMAGE ASSESSMENT OF BUILDINGS AFTER EARTHQUAKES

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Abstract

Damage assessment of structures after extreme events such as earthquakes is an essential and critical task for owners, users, authorities and the community. Accurate and quick damage assessment of structures can effectively reduce economic losses and speed up the reconstruction of the affected earthquake region. Visual inspection is currently the most common damage assessment technique. However, this technique is subjective, time consuming, dangerous for inspectors and not reliable for complex and large structures. Following this approach, building owners need to wait in line for their buildings to be visually inspected and tagged by city officials or evaluated by an engineer in order to assess the status of their building. This process may take days, weeks or even months due to the large number of buildings requiring inspections and evaluations. Given these shortcomings in visual inspection, significant research has been carried out by researchers over the past several years to determine the feasibility of vibration based damage assessment of instrumented structures and establish a coherent and consistent set of techniques and methodologies of real-time damage detection and performance evaluation. In this paper, new computational tools for damage identification and long-term dynamic monitoring, developed in the MATLAB environment, are described. The toolkits provide functions for automated dynamic parameters and response amplitudes monitoring. The potential of these toolkits is illustrated using data collected by a continuous dynamic monitoring system installed on a building in Wellington, New Zealand.

Keywords: Structural health monitoring; Damage detection; Structural dynamics; Modal parameters; Earthquakes

1. Introduction

Several civil engineering structures including bridges, buildings and tunnels continue to be used despite aging and the associated risk of damage accumulation. Therefore, monitoring the structural integrity of these structures is becoming increasingly important from both economic and life-safety viewpoints. Civil engineers in charge of safety and maintenance of these structures are aware of the limitations of their current common practice of condition assessment based on visual inspections [1]. Routine condition assessment is carried out on structures on various intervals. The consequence could be sudden collapse between inspection intervals and unbearable costs on governments and owners for replacement and retrofit tightened up by shrinking budgets. The expressed intention of the building owners globally is to assess structures integrity, strength and performance through the application of sophisticated methods based on actual measurements [2, 3].

Moreover, damage assessment of structures after extreme events such as earthquakes is an essential and critical task for owners, users, authorities and the community. Accurate and quick damage assessment of structures can effectively reduce economic losses and speed up the reconstruction of the affected earthquake region. Utilization of a sensor network system integrated within the structure itself can greatly enhance the inspection process through rapid in-situ data collection and processing. However, these sensor networks typically produce large and complex sets of data that become difficult to process using on-hand database management tools or traditional data processing applications. The challenges include capture, storage, search, sharing, transfer, analysis and visualization.

Several structural health monitoring (SHM) systems implemented in New Zealand and worldwide typically continuously record and store low level acceleration data that is never utilized or even accessed. Moreover, despite recent significant advances in sensor technology hardware, data transmission techniques and computer processing capability, the development in algorithms to fully utilize these advances has not been at a comparable level. The main objective of this project is to advance the existing science to support the development of online structural safety expert systems, which utilize building instrumentation data to pinpoint and track damage progression during operational life and shortly after extreme events. The ultimate goal of this research is to design and develop an automated damage identification system for continuous health monitoring of civil engineering structures. In this automated system, a cluster of computers will perform different steps of the damage detection process, including control of equipment and hardware, data collection, data analysis and generation of evaluation reports and triggering an alarm signal in the case of damage detection. This system can be an essential tool not only for damage detection and SHM of structures but also to manage data from several monitored structures.

In this paper, new computational tools for damage identification and long-term dynamic monitoring, developed in the MATLAB [4] environment, are described. The toolkits provide functions for automated dynamic parameters and response amplitudes monitoring. The application of these toolkits is illustrated using data collected by a continuous monitoring system installed on a building in Wellington, New Zealand.

2. Long-term dynamic monitoring

2.1 Dynamic behavior

Each civil engineering structure has a unique dynamic behavior which may be addressed as a ‘dynamic signature’. This ‘dynamic signature’ is typical for a structure and can be obtained by appropriate measurements and used for the evaluation of the condition and performance of the structure and detection of damage after respective assessment.

Structure ambient vibration induced by wind, traffic, tremors and operational use can be recorded periodically at user specified intervals, continuously, or based on pre-trigger settings. Dynamic characteristics of structures and key response parameters such as peak acceleration, root mean square (RMS) of response and many others can be extracted from the recorded raw data and then interpreted to evaluate the structure’s ‘dynamic signature’.

Ambient vibration based evaluation of structures is selected in this study for building condition assessment under the premise that it can be used practically without any impairment of the building functionality. One of the main goals of the developed system is to provide a monitoring system that makes it possible to reduce the employment of bulky inspection equipment by well-aimed specification of suspected damage zones, therefore minimizing the disturbance to functionality during inspection works.

By measuring the actual dynamic characteristics, the ‘dynamic signature’ is obtained and is not subject to the circumstances of the personnel carrying out the test. Therefore, the condition assessment of the monitored structure can be determined by a systematic analytical evaluation.

2.2 Automated monitoring system

Utilization of a sensor network system integrated within the structure can greatly enhance the inspection process through rapid in-situ data collection and processing [5, 6]. However, these sensor networks typically produce large and complex sets of data that it becomes difficult to process using on-hand database management tools or traditional data processing applications [7-9]. The challenges include capture, storage, search, sharing, transfer, analysis and visualization. The main objective of this research is to design and develop new computational tools for structural modal identification and long-term dynamic monitoring. The toolkit presented here is developed in the MATLAB environment as it permits the easy development of graphical interfaces and provides powerful tools for automated data processing; a characteristics that is essential in the context of continuous monitoring. The developed system consists of two independent toolkits: the modal parameters identification toolkit (MPIT), which is used for structural dynamic identification [10], and the automated data analysis toolkit (ADAT), which is used for data management and processing of large data sets.

The MPIT is mainly used for identification of dynamic characteristics such as natural frequencies, mode shapes and damping ratios [11-13]. In this toolbox, frequency domain based and time domain based system identification techniques are implemented. System identification techniques also include output only as well as input-output methods. The toolbox offers extensive functionalities for the visualization and processing of the data, the determination and visualization of the structure’s modal parameters, comparison of the identified modal parameters from different modal parameters identification techniques and comparison of modal parameters obtained from a particular technique using data from different data sets. The intention is to develop an automated toolbox for estimating modal parameters and providing functions to compare the system identification results from various available techniques and also to compare modal parameters produced by a specific system identification technique using different data sets.

The ADAT is used for automated dynamic monitoring, excluding any user interaction. This toolkit can be utilized to manage and process large data sets automatically. Available tools in this toolkit include functions for communications with servers, data downloads, changing the format of the raw data, dividing continuous data to user specified intervals, running data analysis and saving the results. The toolkit also provides functions for data visualization and comparison. A panel in ADAT includes functions for running modal analysis, identification of dynamic features and estimation of power spectral density (PSD) using recorded data intervals. Modal analysis function in this panel runs selected system identification techniques in the MPIT toolbox. Previously saved results from any structure in a specific time frame can be downloaded and visualized using several available functions in another panel.

The potential of the developed system is illustrated using data collected from a building by a continuous dynamic monitoring system for nearly one year. Ambient vibration data were continuously recorded using 15 tri-axial accelerometers at a rate of 50 samples per second for several years. Acceleration data between 28 January and 20 October 2013 were considered in this study as two major earthquakes and several aftershocks were recorded during this period.

3. Test building

3.1 Structure

The test building in this investigation is located in Wellington, New Zealand's capital city. The building is located in area of high seismicity as the dominant earthquake source in New Zealand, the Wellington Fault, passes close to the city center.

Due to the importance of the building and the high seismicity of the building location, it has recently been instrumented with strong motion accelerometers. The building is instrumented with tri-axial accelerometers spread throughout the structure on various levels. Acceleration data from the test building has been recorded continuously at a rate of 50 samples per second for several years. Acceleration data between 28 January and 20 October 2013 were considered in this study as two major earthquakes and several aftershocks were recorded during this period.

3.2 Strong earthquake events recorded during monitoring period between 28 January and 20 October 2013

3.2.1 Seddon earthquake

The M_W 6.5 Seddon earthquake struck at 5:09:30 pm on 21 July 2013 (05:09 UTC) at a depth of 13 kilometers and was centered around 55 kilometers south of Wellington, according to GeoNet [14]. The quake caused moderate damage in and Wellington and nearby cities. Only minor injuries were reported and several aftershocks occurred after the earthquake.

The earthquake was preceded by a series of foreshocks and generated a series of aftershocks. Table 1 lists all foreshocks and aftershocks of M_L 5.0 and above that occurred in the region between 19 July 2013 and 2 August 2013 [14].

Table 1 – Summary of strong earthquake events recorded between 19 July 2013 and 2 August 2013.

Date (NZST)	Time (NZST)	Magnitude (M_L)	Epicenter	Depth
19 July 2013	9:06:39 am	5.7	30 km east of Seddon	17 km
21 July 2013	7:17:10 am	5.8	30 km east of Seddon	20 km
21 July 2013	**5:09:30 pm**	**6.5**	**25 km east of Seddon**	**13 km**
21 July 2013	5:13:50 pm	5.2	30 km east of Seddon	13 km
29 July 2013	1:07:14 am	5.4	20 km east of Seddon	12 km
2 August 2013	12:56:13 am	5.2	20 km east of Seddon	6 km

3.2.2 Lake Grassmere earthquake

The M_W 6.6 Lake Grassmere earthquake occurred at 2:31:05 pm (NZST) on Friday 16 August 2013 [14]. The epicenter was located about 10 km south-east of Seddon city, under Lake Grassmere, with a focal depth of 8 km. The earthquake caused significant damage buildings in Seddon and it was widely felt in both the North and South Islands of New Zealand. The earthquake generated a significant series of aftershocks, the largest was M_L 6.0. Table 2 lists of all aftershocks M_L 5.0 and above that occurred in the region between 16 August 2013 and 5 September 2013 [14]. A summary of the vertical and horizontal peak ground acceleration records from Seddon and Lake Grassmere earthquakes are shown in Fig. 1 (a) and (b), respectively [14]. As can be seen in these figures, peak ground accelerations produced by the two major earthquakes and their associated aftershocks were similar in Wellington region.

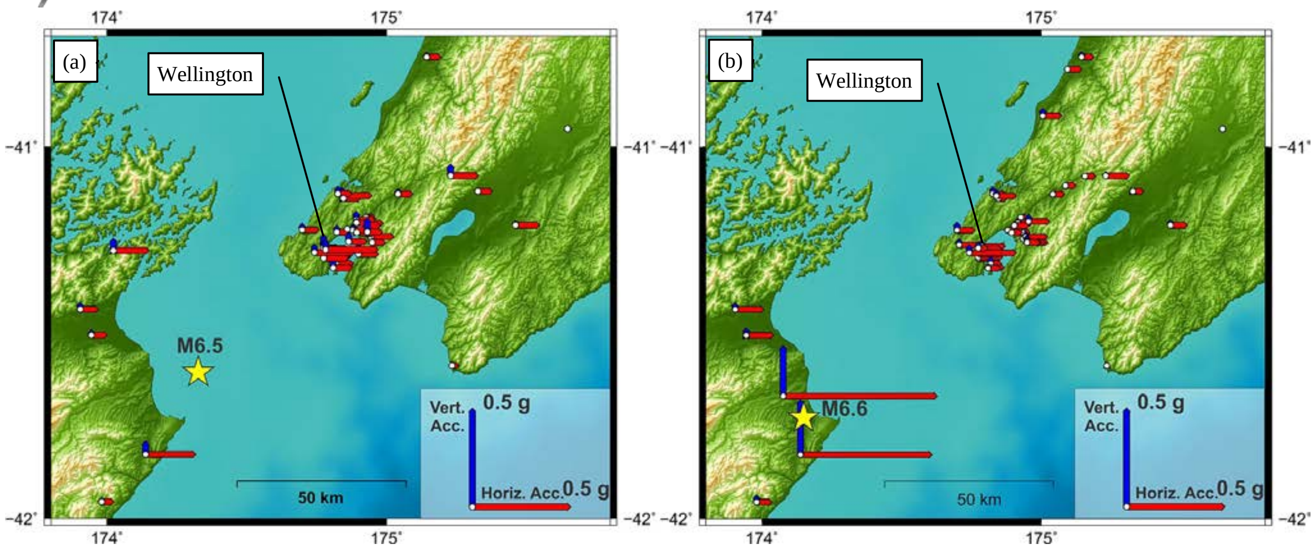

Fig. 1 – Summary of peak ground accelerations during the (a) Seddon earthquake (b) Lake Grassmere earthquake [14]

Table 2 – Summary of strong earthquake events recorded between 16 August 2013 and 5 September 2013.

Date (NZST)	Time (NZST)	Magnitude (M_L)	epicenter	Depth
16 August 2013	**2:31:05 pm**	**6.6**	**10 km south east of Seddon**	**8 km**
16 August 2013	2:37:27 pm	5.4	5 km south east of Seddon	9 km
16 August 2013	2:45:27 pm	5.4	10 km south east of Seddon	6 km
16 August 2013	2:56:27 pm	5.0	20 km east of Seddon	9 km
16 August 2013	3:09:08 pm	5.5	10 km south of Seddon	8 km
16 August 2013	3:21:31 pm	5.0	10 km south of Seddon	17 km
16 August 2013	3:51:35 pm	5.6	10 km east of Seddon	19 km
16 August 2013	5:31:16 pm	6.0	15 km east of Seddon	14 km
16 August 2013	5:56:10 pm	5.0	5 km northwest of Seddon	10 km
16 August 2013	5:57:52 pm	5.1	20 km east of Seddon	5 km
16 August 2013	6:42:40 pm	5.2	20 km east of Seddon	20 km
16 August 2013	6:55:58 pm	5.5	20 km east of Seddon	20 km
16 August 2013	8:38:54 pm	5.2	10 km southeast of Seddon	22 km
17 August 2013	4:13:20 pm	5.0	15 km southwest of Seddon	20 km
17 August 2013	8:58:39 pm	5.5	10 km south of Seddon	20 km
18 August 2013	4:07:52 am	5.0	5 km southeast of Seddon	20 km
5 September 2013	12:04:10 am	5.1	35 km northeast of Seddon	16 km

4. Results

4.1 Data management and processing

Acceleration data from the instrumented building were recorded continuously at a rate of 50 samples per second and the raw data were stored to a remote server. Given the high sampling rate (50Hz), the large number of recording channels and the long monitoring period considered in this study (268 days), the SHM system produced a very large amount of monitoring data that could be very challenging to manage, process and analyze. In the considered monitoring period from 28 January to 20 October 2013, the SHM system produced more than 19,296 (268 days x 24 hours x 3 directions) hourly interval files with around 699 GB of total data size. The developed toolkit in this study was successfully utilized to transfer the raw data from the remote server, change the data format, separate the acceleration data in different directions into individual files, divide the data to hourly intervals and then process each interval and report the results in a fully automated way. Data analysis was carried out in three stages. Firstly, a succession of PSD lines was produced to form a spectrogram of hourly distribution of frequency components. Secondly, an automated modal parameters identification procedure was implemented to extract dynamic characteristics from successive hourly data sets. Then, peak and RMS accelerations were detected automatically, enabling the statistical treatment of the response time series.

4.2 PSD spectrogram

The developed toolbox performs automated frequency-domain analysis of acquired data, evaluating the PSD spectra at different sensors. By plotting sequences of the PSD estimates of every hour of data, spectrogram plots are obtained, as shown in Figs. 2-7. From spectrogram plots, the frequency component distribution is easily captured, allowing the observation of the time variation of natural frequencies, as well as the identification of different intensity periods.

PSD estimates of every hour of data is plotted versus the time and date of the recorded data. PSD amplitude of a particular frequency at a particular time is represented by the line color, with dark blues corresponding to low amplitudes and brighter colors up through red corresponding to progressively stronger amplitudes. Figs. 2 to 7 depict the PSD distribution of horizontal acceleration data obtained from a number of accelerometers located at various locations in the building, in the period between 28 January and 20 October 2013. Fig. 2 shows PSD distribution of horizontal acceleration in the East-West direction from accelerometer #1 which is located at the top of the building. The time of the two major earthquake occurred during the monitoring period is indicated by two red lines at 2013.07.21 and 2013.08.16. As clearly seen in this figure, a drop of 0.05 Hz in the 1.66 Hz natural frequency has been observed immediately after the Seddon earthquake (21 July 2013) indicating a very minor but permanent alteration of the building dynamic performance due to this earthquake. No further drop or change in the natural frequency was noticed after the second major earthquake; Lake Grassmere earthquake which occurred on 16 August 2013. To ensure that this drop in frequency is not due to equipment malfunction or any sensor related issue, PSD distribution using data from different sensors were also inspected (Figs. 3 and 4). Similar frequency shifts can be seen in these figures eliminating any effect of equipment malfunction.

PSD data from horizontal acceleration records in the North-South direction indicate similar drop in the natural frequency of a number of modes after Seddon earthquake, as shown in Figs. 5-7. Another important observation was made by comparing modal amplitudes in the East-West direction to that in the North-South direction. Comparison of Figs. 2-4 to Figs. 5-7 respectively show that modal amplitudes of the first translational mode increased in the East-West direction and decreased in the North-South direction after the two major earthquakes and their associated aftershocks indicating a small shift in the vibration direction of the first translational mode from North-South to East-West.

4.2 Modal parameters

As explained in previous sections, modal parameters including natural frequencies, mode shapes and damping ratios can be estimated using eight system identification techniques available in MPIT program. In this study, SSI was selected to perform modal analysis as it is one of the most robust and reliable techniques although this method is more computationally demanding than other available techniques in MPIT. For each set of hourly data in a specific vibration direction, the system identification analysis run time was approximately five minutes. The total

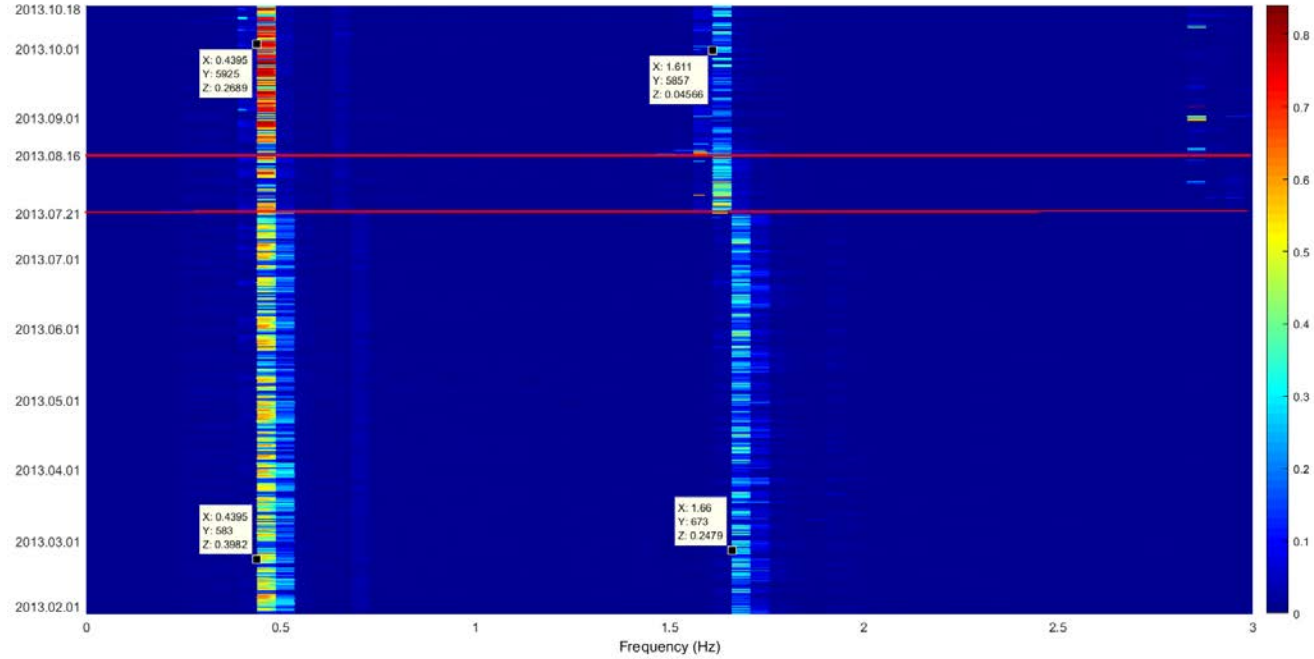

Fig. 2 – PSD (Acc. #1) 28Jan-20Oct-2013 East West Direction

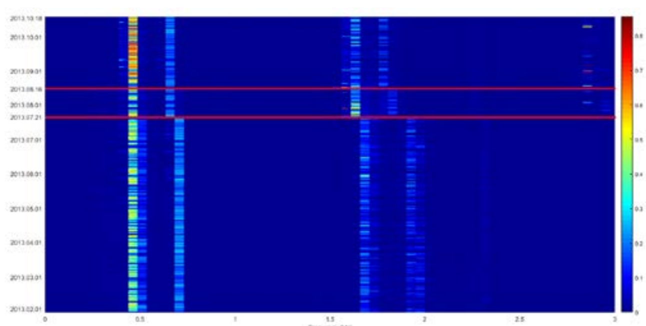

Fig. 3 – PSD (Acc. #2) East West Direction

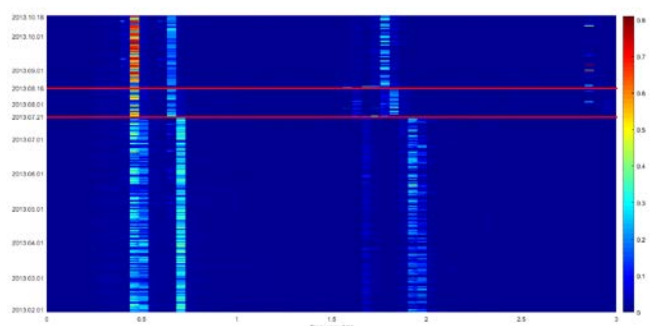

Fig. 4 – PSD (Acc. #3) East West Direction

analysis time for each vibration direction was therefore around three weeks (268 days x 24 hours x 1 direction x 5 minutes). This extensive, repetitive and lengthy analysis emphasizes the need for developing an automated system for processing the data. Achieving reliable results and avoiding human errors in processing large data sets was the second main goal of developing the proposed toolkits in this study. Fig. 8 depicts natural frequencies and the corresponding damping ratio versus the number of times of detecting the pair values in the vertical axis. The intention here is to monitor variations in the building modal characteristics due to natural sources such as existence of noise and seasonal changes in temperature throughout the building design life. Accurate evaluations of variations in dynamic modal characteristics of structures is an essential step for establishing a reliable threshold of these parameters for the purpose of damage detection and structural health monitoring. As can be seen in Fig. 8, variation in damping ratios is relatively large compared to that in natural frequencies due to the fact that the estimation of modal damping ratio from ambient vibrations data is the most challenging task in system identification.

4.3 Vibration intensity

The vibration intensity is a good indicator for the stress level of a structure subjected to dynamic loads. Increasing vibration intensities of individual structural members under similar operational loads can be an indicator of fatigue-

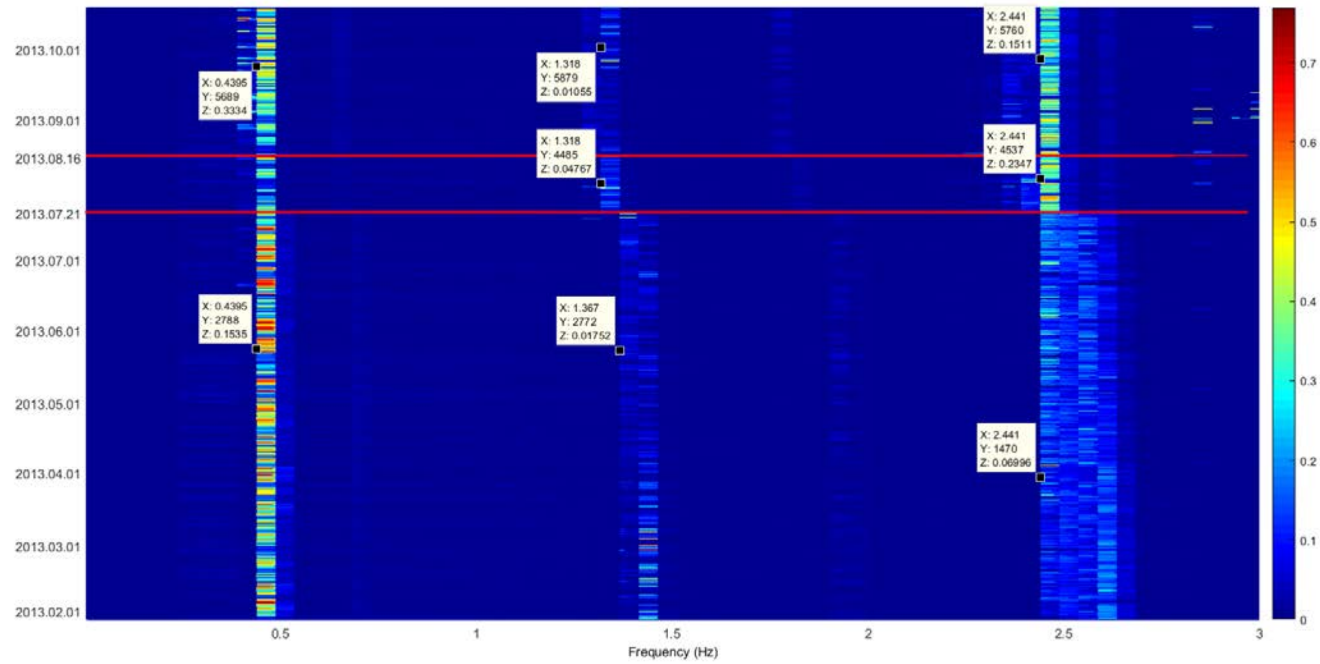

Fig. 5 – PSD (Acc. #1) 28Jan-20Oct-2013 North South Direction

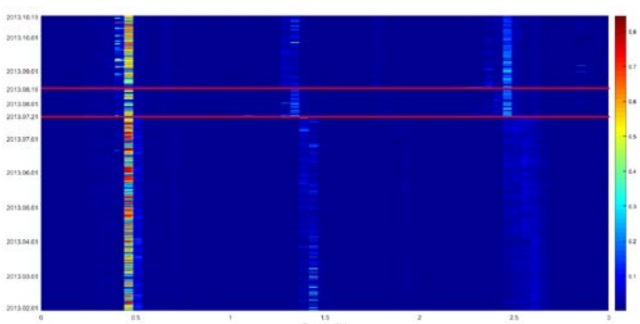

Fig. 6 – PSD (Acc. #2) North South Direction

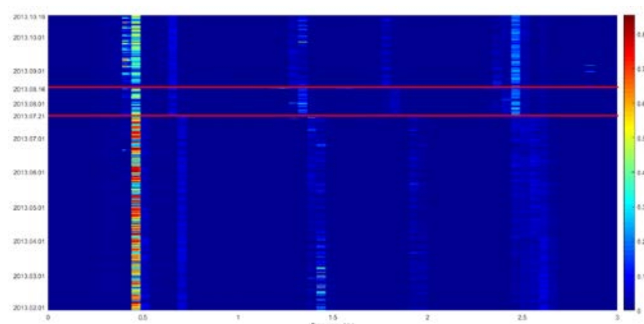

Fig. 7 – PSD (Acc. #3) North South Direction

relevant damage mechanisms. Raw horizontal acceleration data was divided to hourly intervals and peak acceleration and RMS of acceleration values were determined for every hour. The frequency of peak acceleration and RMS values were then estimated to construct histograms of these parameters.

Figs. 9 and 10 show histograms of the peak acceleration values reported from the accelerometer at the roof level of the structure for the monitoring periods between January-June 2013 (before earthquakes) and August-October 2013 (after earthquakes), respectively. The intention here is to compare the vibration intensities before and after the two major earthquakes including several strong aftershocks. Peak acceleration values in the range of 0 to 0.02 m/s^2 were used to plot the histograms. Both peak acceleration histograms before and after earthquake series showed skewed distribution but with different mode values. Comparison of the histograms in Figs. 9 and 10 indicate a small increase in the peak acceleration mode value from 0.00365 m/s^2 before the earthquakes to 0.00587 m/s^2 after the earthquake series. It is not possible to relate such increase in the mode value to a drop in the building stiffness due to the lack of data points after the earthquakes. More acceleration data and detailed information about wind speed, distribution and direction, weather conditions and operational loads are needed to be able to correlate the change in peak acceleration histograms to the building stiffness with enough confidence.

RMS of horizontal acceleration data before and after the earthquakes are shown in Figs. 11 and 12, respectively. Similar distribution and amplitudes of RMS data can be clearly observed in the two monitoring periods. Similarity in the RMS distribution and amplitudes indicates to a good extent that the building performance has not been altered after the earthquakes. The sensitivity of vibration intensity indicator such as peak acceleration and RMS to small damage levels needs further investigation.

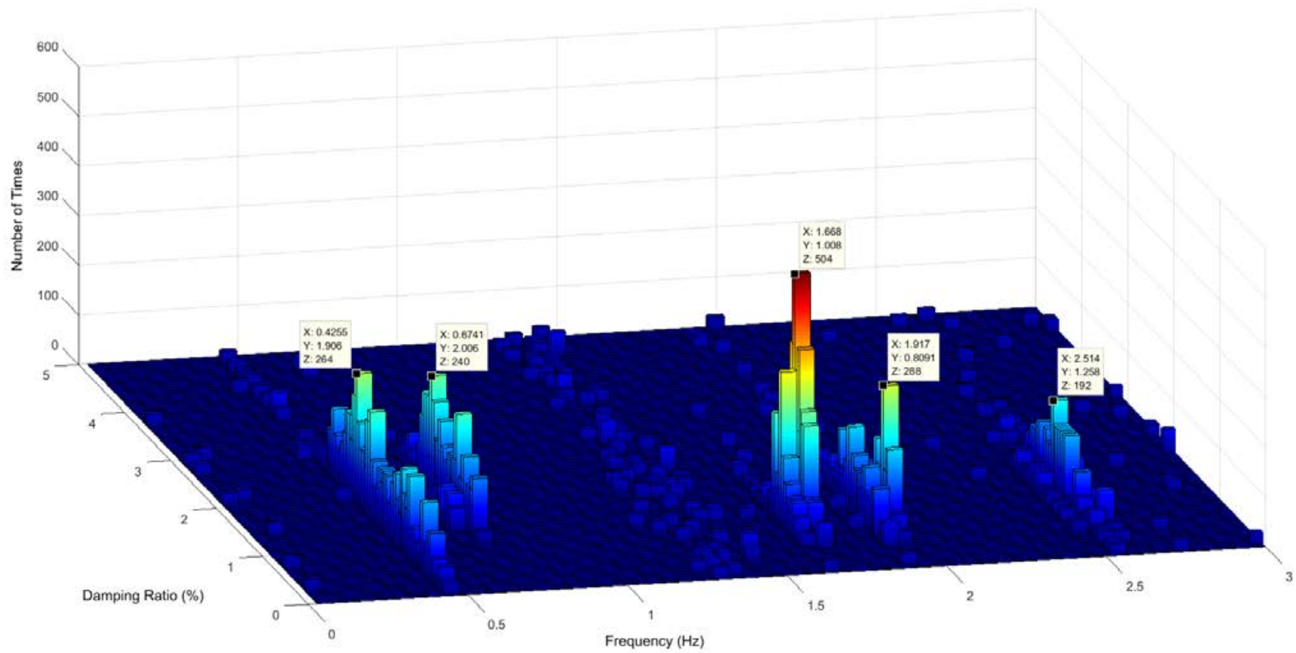

Fig. 8 – Histogram of Frequency and damping ratio

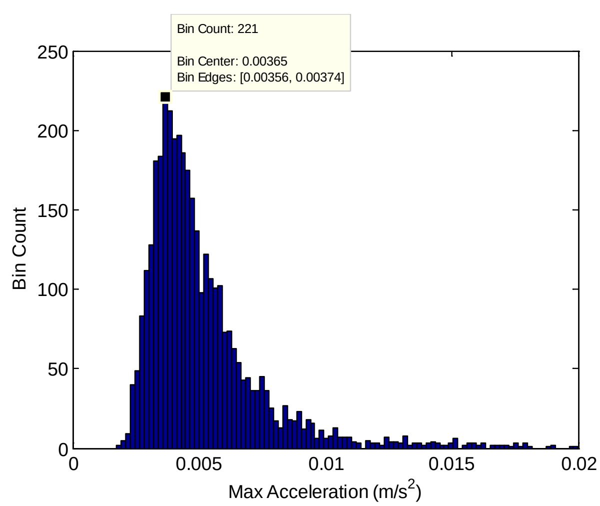

Fig. 9 – Peak Acceleration Data 28Jan-30June-2013 (Before Earthquakes)

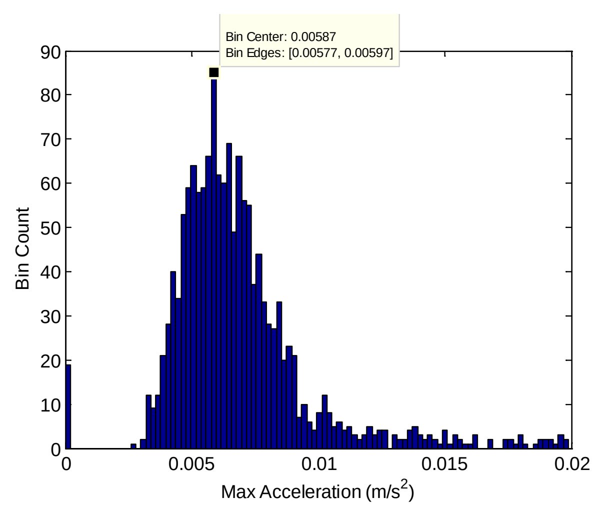

Fig. 10 – Peak Acceleration Data 17Aug-20Oct-2013 (After Earthquakes)

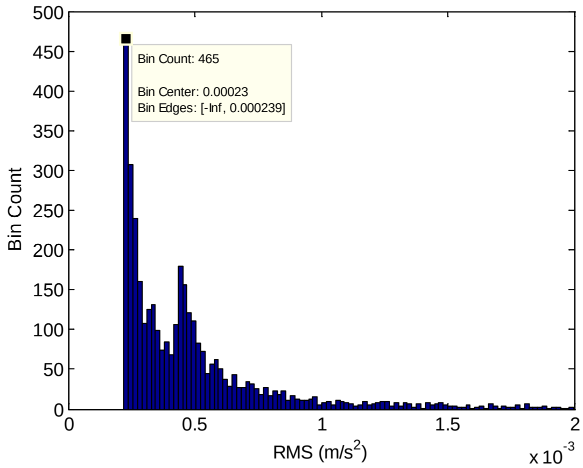

Fig. 11 – RMS of Acceleration Data 28Jan-30June-2013 (Before Earthquakes)

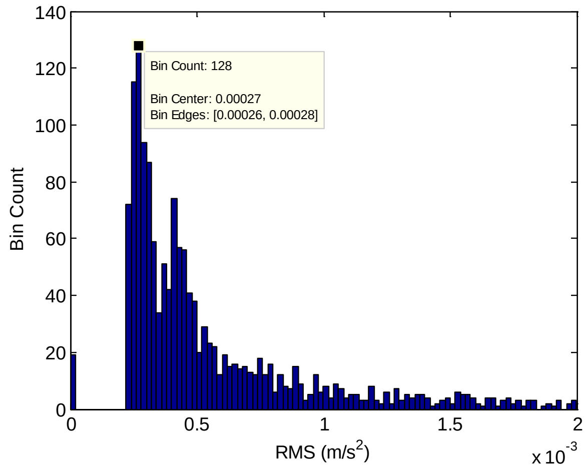

Fig. 12 – RMS Acceleration Data 17Aug-20Oct-2013 (After Earthquakes)

5. Conclusion

The applications of a new monitoring toolkit in the context of long-term monitoring has been demonstrated using acceleration data recorded over several months from a full scale building located in Wellington, New Zealand. The building was subjected to two major earthquakes of M_W 6.5 and 6.6 and several aftershocks during the monitoring period.

The dynamic performance of the building was evaluated before and after the earthquakes utilizing the developed toolkit. A very small drop in the natural frequency of few modes was observed after the first earthquake but with no major change in the overall dynamic performance of the building. No further drop or change in the natural frequency was noticed after the second major earthquake. Modal amplitudes of the first translational mode increased in the East-West direction and decreased in the North-South direction after the two major earthquakes and the associated aftershocks indicating a shift in the direction of the building vibration in the first mode toward the East-West direction.

A small increase in the peak acceleration mode value was observed after the earthquake series but this cannot be immediately correlated to a change in the structure conditions due to the lack of data and characteristics of operational loads. Similar distribution and amplitudes of RMS acceleration data were observed before and after the earthquakes sequence indicating no change in the structure overall stiffness.

The developed toolkit in this study was successfully utilized to manage, analyze and report the results from very large sets of recorded data from a continuous SHM system in a fully automated way. The effectiveness of the toolkit in running computationally extensive analysis has also been demonstrated.

This investigation illustrates the potential of this package in terms of automated processing of large amount of data enabling the accurate characterization of the time variation of natural frequencies and dynamic performance indicators over long periods of continuous monitoring.

6. Acknowledgements

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