Monitoring of lateral stiffness changes for a real building using hysteresis loop analysis

M. Rabiepour, J.G. Chase & G.W. Rodgers
University of Canterbury, Christchurch.

C. Zhou
Northwestern Polytechnical University, China.

ABSTRACT

The BNZ building in Wellington, New Zealand was significantly damaged, and eventually demolished, due to the cumulative impact of three main earthquakes between 2013 and 2016. Although the building was equipped with a set of accelerometers, lack of an accurate structural health monitoring (SHM) method allowed the damage level to remain hidden to engineers and property owners, according to the reports published after the damaging seismic events. An accurate SHM method would have identified the structural damage location and extend in this building over each of three major events, enabling proper retrofit and better decision making.

This study uses the hysteresis loop analysis (HLA) SHM method to investigate lateral stiffness changes for this structure, and the trajectory of damage accumulation for each storey over all three events. HLA indicates the building suffered average stiffness reductions of 21%, 20% and 8% due to each event, respectively. The first storey lost ~67% of its initial stiffness by the end of the third event. Results show a very good consistency between events over all 5 storeys, demonstrating accurate and continuous performance of HLA in capturing lateral elastic stiffness trajectories.

HLA shows very good robustness and consistency in estimating lateral elastic stiffness changes due to damage in this real-world, full-scale case analysis. Consistent, accurate estimation can help optimal decisions after major events. They also offer the future potential to build a virtual building model accurately predict forward what might happen to an already damaged structure in future events.

1 INTRODUCTION

Civil structures are unpredictably subjected to damaging earthquakes, adversely affecting their future performance (Erazo 2019). Post events, accurate evaluation of structural integrity plays an important role in minimizing economic and human life losses for future events. Moreover, precise damage information helps optimize resources and strategies required to retrofit affected structures to best ensure life safety and service (Erazo 2019).

SHM evaluates structural status by detecting the presence, location and severity of structural damage by analysing structural response induced by external excitations (Entezami 2019). Typically, an SHM system consists of three main parts: 1- a sensory system measuring structural responses; 2- a data processing system consisting of data transmission, processing and storage; and 3- a damage identification algorithm analysing measured responses to evaluate structural damage (Abdulkarem 2020). Deficiency in any of these three parts reduces SHM performance. A robust, accurate, and easy-to-use or easily automated SHM algorithm is essential for precise SHM after significant events.

Despite significant research, SHM is not practically implemented in the real world. European Joint Research Centre (JRC) reports show a considerable gap between SHM research and application (Gkoumas 2019). Only a few SHM algorithms have passed experimental testing on scaled or full-scale structures (Chen 2017, Lei 2020, Wang 2020a, Wang 2020b, Yang 2020, Zhou 2017c), and very few have been tested for full-scale real-world buildings and infrastructure (Akhlaghi 2019, Bulajić 2020, Chen 2017, Gallipoli 2020, Nguyen 2019, Rahmani 2015, Zhou 2017b). In real-world cases, excitations were often not strong enough for structures to be excited in a fully nonlinear range and experience significant damage, thus limiting proof of SHM efficiency.

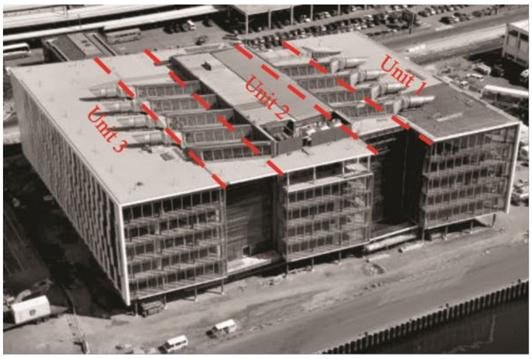
This paper presents SHM of an actual instrumented building, significantly damaged under multiple seismic events. The theoretically (Xu 2014, Xu 2015, Zhou 2017a) and experimentally (Zhou 2019, Zhou 2017b, Zhou 2017c) validated hysteresis loop analysis (HLA) SHM method is employed based on it is demonstrated accuracy and continuity over multiple events and response types (Rabiepour 2020, Zhou 2017a, Zhou 2018). HLA has recently been used to clone a predictive, SHM-based structural model, or digital clone, predicting collapse (Fitzjohn 2020, Zhou 2020a, Zhou 2020b) offering the potential to extend accurate real-world SHM further and more valuably into post-event decision making.

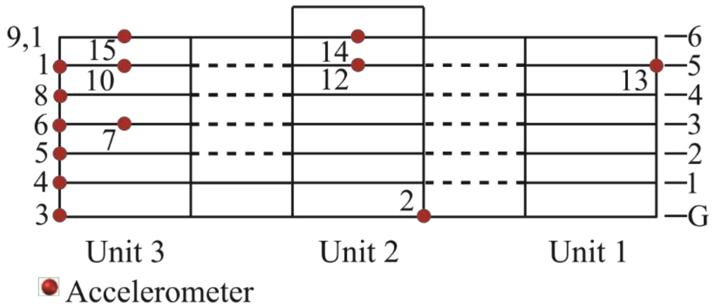
The real case study employed in this paper is the former BNZ office building situated in Wellington, New Zealand. This building was demolished in 2019 due to the severity of damage induced by a series of damaging earthquakes between 2013 and 2016 (Chandramohan 2017).

2 CASE STUDY

2.1 BNZ structure details and instrumentation

The former BNZ building in Wellington, New Zealand was a modern structure opened in 2009 (Fig. 1a). It was a five-storey RC building supporting an extra steel-frame storey on its roof. Although all references described the building consisted of three separate units linked together by pedestrian bridges (Tan 2018, Uma 2010), the authors found Units 2 and 3 are horizontally and effectively linked via a floor diaphragm at level 1 (Fig. 1b), as presented in (Uma 2010). This linkage causes the first storey to be stiffer compared to the other storeys. The building was instrumented using 15 tri-axial accelerometers in 2010, as a part of the EQC funded GeoNet Monitoring Program (Uma 2010). Figure 1b shows Unit 3 in the BNZ building is fully instrumented, with at least one accelerometer on each storey. Sensors 3, 4, 5, 6, 8, 10 and 15 are used for levels G to 6 in the building, respectively. These sensors monitor the motion at the centre of the indicated levels. Apart from Sensor 3, which was a floor-mounted sensor installed on the floor of the Ground level, the other sensors were ceiling-mounted sensors attached to the floor-support beam crossing the centre of the relevant level (Uma 2010).





(a) **(b)**

Figure 1: (a) The aerial photograph showing the BNZ Building. (b) Schematic showing the location of installed accelerometers in the BNZ building.

2.2 Damaging earthquakes

The 2013 Seddon earthquake required closure of the BNZ building for 15 months for repair (SCHOUTEN 2014). Only some non-structural damage in the top floors was reported, and the building was scheduled to be reopened in ~6-8 weeks (Stuff 2013). Four weeks after the Seddon event, during the closure time, the 2013 Lake Grassmere earthquake occurred. It was reported the second event did not have any significant further effects on the building (Vaughan 2014). In 2016, after ~2 years of reoccupation, the 2016 Kaikoura earthquake caused significant visible structural and non-structural damage, including cracks in concrete members, falling ceiling panels, and broken glass facades (Chandramohan 2017, Rutherford 2016).

Table 1: Details of the three damaging earthquakes.

Event	Earthquake	Occurrence time	Magnitude (M_w)	Distance* (km)
1	Seddon	05:09 pm – 21/7/2013	6.5	50
2	Lake Grassmere	02:31 pm – 16/8/2013	6.6	70
3	Kaikoura	12:02 am -14/11/2016	7.8	215

*Distance between the earthquake's epicentre and the BNZ building's site location is reported.

Table 1 lists details of these three key events. The moment magnitude scale (M_w) is used to report the size of these large, destructive seismic events. As can be seen, the third earthquake had a bigger magnitude and occurred at a considerable farther distance from the BNZ building site location compared to the other two earthquakes. Fortunately, the first and third earthquakes happened after working hours, when the BNZ building was relatively unoccupied. At the time of these two destructive earthquakes the building had its own normal operation while it was vacated for repair during the second event (Chandramohan 2017).

Figure 2 shows the ground acceleration records, monitored by the sensor 3 installed at Level G in Unit 3 (Fig. 1b), at the BNZ building site. The absolute values of the measured peak ground acceleration (PGA) are 0.40, 0.15 and 0.27 for the three events, respectively. Stages I-IV in Figure 2 are defined to report the identified initial and final elastic stiffness for each event.

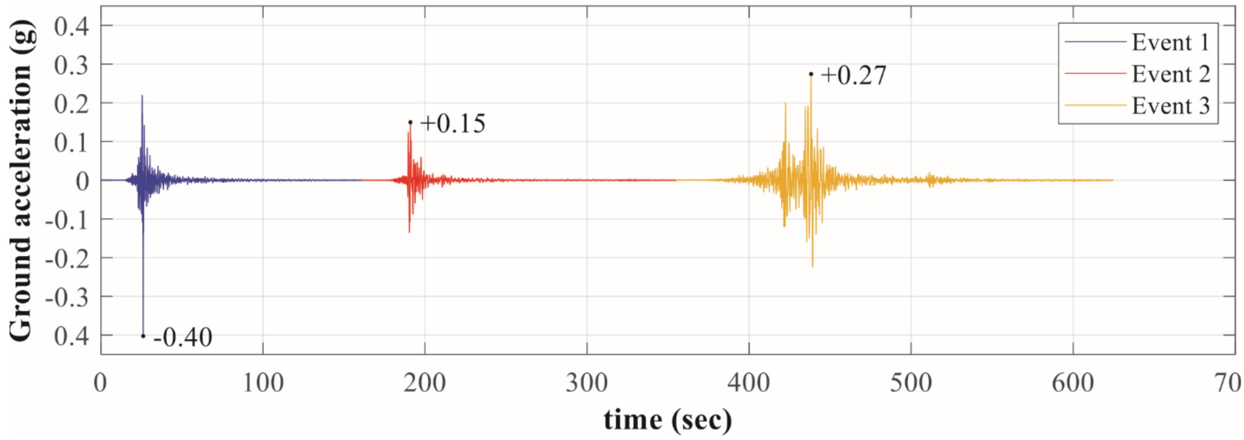


Figure 2: The ground shaking acceleration measured at the BNZ building's site location by sensor 3 for the three events.

3 HLA SHM METHOD

The well-developed HLA SHM method accurately estimates lateral stiffness changes using linear regression and statistical hypothesis testing. It identifies storey stiffness and its evolution using measured hysteretic force-deformation loops divided into component half cycles with loading and unloading branches (Xu 2015, Zhou 2015b). To form these hysteresis loops, inter-storey restoring force, f, and inter-storey displacement, x, must be obtained for each storey, or groups of storeys, of the monitored building. Figure 3 shows in summary how these hysteresis loops are reconstructed in HLA. Several studies have shown the robustness, accuracy and consistency of HLA across multiple events (Zhou 2017a, Zhou 2015a). Full details of HLA are found in (Zhou 2017a, Zhou 2015b).

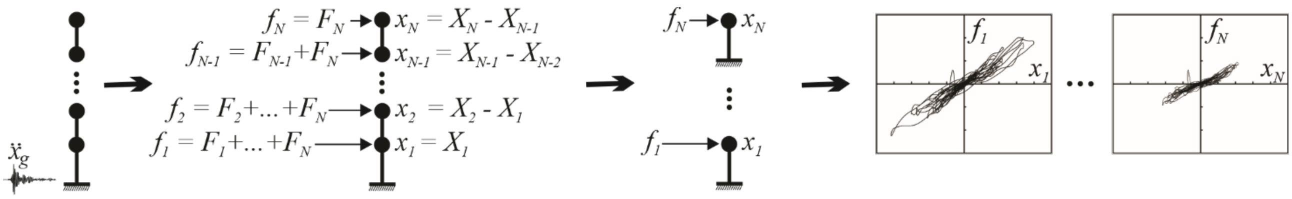


Figure 3: The required steps to calculate force-deformation hysteresis loops in HLA-SHM method.

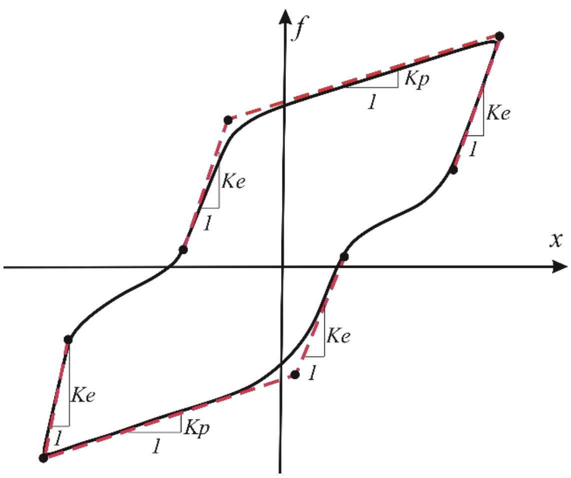


Figure 4: Schematic showing how HLA captures elastic stiffness, K_e, for a pinched hysteresis loop.

Figure 4 displays schematically how HLA identifies elastic stiffness, K_e, and post-yielding stiffness, K_p, from a pinched hysteresis force-deformation loop. This type of nonlinear behaviour is quite typical in RC-frame buildings and also observed in the BNZ building responses under the three damaging seismic events. In this paper, elastic stiffness changes, as the main measure for structural damage, are evaluated using HLA SHM method for the building under the three earthquakes.

Since Unit 3 of the BNZ building is fully instrumented (Fig. 1b), it is feasible to calculate force-deformation hysteresis loops for each storey of this unit. To obtain inter-storey restoring force for each storey, f_i, the following equations are used:

$$M\ddot{X}(t) + C\dot{X}(t) + K(t)X(t) = -MI\ddot{x}_g \quad (1)$$

$$F(t) = K(t)X(t) = -M\ddot{X}(t) - C\dot{X}(t) - MI\ddot{x}_g \quad (2)$$

$$f_i(t) = \sum_{j=i}^{N} F_j(t) \quad (3)$$

where, X , $\dot{X}$ and $\ddot{X}$ are displacement, velocity and acceleration responses of the building subjected to ground acceleration $\ddot{x}_g$. The mass, M, and damping, C, matrices are constant while the stiffness matrix, $K(t)$, changes with time due to earthquake-induced structural damage. In equation 3, N is the number of storeys of the building. To obtain inter-storey displacement, x, the double integration technique is used to calculate displacement from the storey acceleration responses recorded directly by the installed accelerometers (Hann 2009, Skolnik 2010, Wu 2008, Zhou 2015b).

The BNZ building has a total floor area of ~25000 m^2 leading to a total seismic mass of ~20×10^6 kg. It is assumed the seismic mass is distributed evenly across all 3 units and all 6 floors. Thus, the mass of each floor is taken as 10^6 kg. Moreover, a Caughey damping model with 5% damping ration is employed to calculate restoring force, which is a rational choice for an RC-frame structure like the BNZ building (Gatti 2018).

4 RESULTS AND DISCUSSION

Figure 5 shows changes of lateral elastic stiffness, K_e, in the building for the three earthquakes. The stiffness values at stages I-VI are summarized in Table 2, showing small differences between the stiffness of stages II-III, and IV-V.

There is the appearance of some inconsistency between events in Figure 5, particularly for storeys 1 and 3. These increases can be due to retrofit, where no specific documentation was found. However, it is also important to note checking agreement across two uncertain identified values using a relative error of 12% for these measures of displacement (Skolnik 2010) shows these estimated values have intersecting ranges, which means they are in agreement. All other results show very good continuity. This level of continuity has only been demonstrated, to date, by HLA in highly controlled scaled (Zhou 2017c) and full-scale tests (Zhou 2019, Zhou 2017b). These results are the first in an instrumented as-built, real-world structure under large earthquakes.

Damage percentages in Table 3 shows how destructive the first earthquake was for the building. Moreover, it is clear the second event was considerably destructive. On average, the BNZ building was damaged more than 40% due to the first two events, despite reports of only superficial damage (Chandramohan 2017, SCHOUTEN 2014, Stuff 2013, Thomson 2014). The maximum degradation occurred in the first storey with a 67% drop. The average stiffness drops across all storeys were 24%, 21% and 9% for each event, respectively.

The stiffness evolutions in Figure 5 reveals no significant structural retrofit happened in Unit 3 after the first two events. If done, it was not enough to improve its behaviour for the future event. This outcome matches reports of only superficial damage after these two events (Chandramohan 2017, SCHOUTEN 2014, Stuff 2013, Thomson 2014).HLA results show major damage after events 1 and 2 requiring major retrofits. However, the building did not receive them because the severity of damage was not identified and no change is seen in structural stiffness afterward before events two and three. At the end, the building had to be demolished less than 10 years into a normal 40-50-year lifecycle, where several of these years was unoccupied, creating a larger economic loss imposed on owners due to a lack of accurate information on the building status.

Unlike any other SHM methods introduced to date, HLA is an automated method, which is generalizable and applicable in the same way in all cases. Such generalizability and automation should be expected from a reliable SHM, open the opportunity to easily provide accurate, robust SHM widely at low cost.

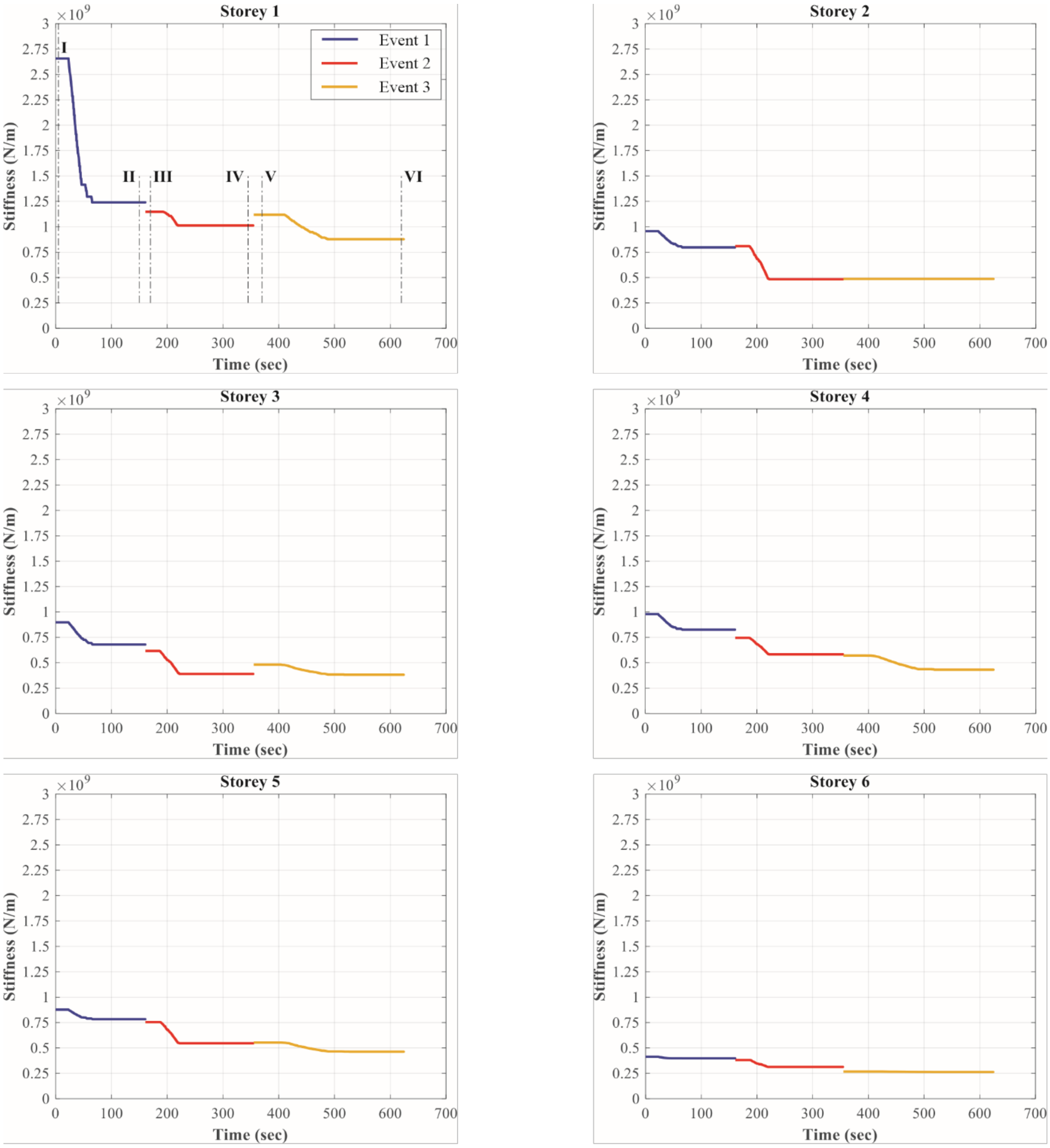


Figure 5: Elastic stiffness evolution captured by HLA for the three events with stages I-VI shown in the plot of Storey 1.

Table 2: Elastic stiffness (× 10^9 N/m) at Stages I-VI for the three events. (NOTE: The shaded areas capture between event continuity and should be approximately equal.)

	Storey					
Stage	**1**	**2**	**3**	**4**	**5**	**6**
I	2.66	0.96	0.9	0.98	0.88	0.41
II	1.24	0.80	0.68	0.83	0.78	0.40
III	1.15	0.81	0.62	0.75	0.76	0.38
IV	1.01	0.48	0.39	0.58	0.55	0.31
V	1.12	0.49	0.48	0.57	0.55	0.27
VI	0.88	0.49	0.38	0.43	0.46	0.26

Table 3: Absolute damage percentage (%) with respect to the initial stiffness (Stage I) for the three events.

	Stages			
Storey	**I-II**	**III-IV**	**V-VI**	**Total Damage**
1	-53	-5	-9	-67
2	-17	-34	0	-49
3	-24	-25	-11	-57
4	-16	-17	-14	-56
5	-11	-24	-10	-47
6	-4	-16	-1	-36

5 CONCLUSIOIN

Results show HLA is robust and accurate in extracting lateral elastic stiffness even for real-world cases subjected to multiple events. The experience of the BNZ building highlights the fact that although efficient instrumenting of buildings for SHM is necessary, but not enough. A reliable SHM method is also required, one was not available or applied to this building, potentially skewing decision making and leading to demolition.

The results show how cumulative damage in each event deteriorated structural performance of the building in the subsequent events, and adversely affected its lifecycle and serviceability. It highlights the importance of a reliable SHM method, such as HLA, after major events to estimate the current state of buildings and predict their future (damage, diagnosis, and prognosis), which was not available for the BNZ building from 2013 to 2018.

This work demonstrates how traditional SHM methods and routine visual inspections, which have been typically being employed for buildings after severe earthquakes, are considerably inaccurate even in identifying structural damage presence. As shown, the unseen structural damage in the BNZ building due to the first two earthquakes in 2013 consequently lead to delayed decisions and a lack of appropriate retrofit actions. More accurate information provided by trustable tested SHM methods, like HLA, accelerates and facilitates decision making and recovery processes, which are vital to society especially aftermath.

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