Structural dynamic response of an unreinforced masonry house using non-destructive forced vibration

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Abstract

The results of non-destructive forced vibration tests on a small-scale unreinforced masonry house with a flexible timber diaphragm are presented. The primary purpose of this study was to investigate the dynamic responses between the as-built and retrofitted structures. This includes assessment of diaphragm response, wall-diaphragm connection details, in-plane wall response, out-of-plane wall response, and the response of wall corners. The test protocols were designed to investigate two types of retrofit techniques consisting of a plywood-retrofit on the diaphragm, and a connection-retrofit between the wall and diaphragm. From the results, one can see that the natural frequency and mode shapes of the first translational mode were affected. The force transfer mechanism of the as-built structure was significantly improved after applying both retrofits whereas each technique shows distinctive enhancements on the structure overall response.

Keywords: unreinforced masonry, structural response, dynamic response, non-destructive, forced vibration

1. INTRODUCTION

Unreinforced masonry buildings have long been recognised to perform poorly in an earthquake. This deficient performance was clearly demonstrated in several major earthquake episodes such as the 1931 Hawke's Bay (Dowrick 1998; Blaikie and Spurr 1992) and 1989 Loma Prieta (Bruneau 1994a; Bruneau 1994b) earthquakes. Due to their poor performance in past earthquakes, an understanding of the structural dynamic behaviour of unreinforced masonry buildings when subjected to seismic excitation is of main interest in many seismically active countries such as the United States, New Zealand, Italy, Portugal and Chile. In New Zealand, the requirement to seismically upgrade these earthquake-prone buildings was mandated by The Building Act 2004 (DBH 2004). Importantly, these buildings form a significant percentage of New Zealand's building stock and represent the predominant national architectural heritage (Russell and Ingham 2008).

Most unreinforced masonry buildings in New Zealand consist of solid unreinforced masonry bearing walls and flexible timber floor and roof diaphragms. The wall thickness configuration over the height of the building was typically reduced by a single leaf at each storey height to support the diaphragm. No connections between walls and diaphragm were observed in the

buildings constructed before the 1931 Hawke's Bay earthquake and majority out-of-plane wall failures were related to the absence of anchorage between the walls and diaphragms. Following the 1931 Hawke's Bay earthquake, most unreinforced masonry buildings were seismically retrofitted, which included the installation of wall-diaphragm connections (Blaikie and Spurr 1992). Most connections applied were through-bolt anchor with the use of a steel bearing plate located on the exterior face of the building walls. The other end of the anchor was identified to be bolted to a steel angle that is commonly bolted to the timber diaphragm members such as floor joist or roof rafters.

In an attempt to assess the structural dynamic responses of existing (as-built) unreinforced masonry buildings in New Zealand, a small-scale unreinforced masonry house was constructed. The main aim of the study was to assess the dynamic responses between the as-built and retrofitted structures. This includes assessment of diaphragm response, wall-diaphragm connection details, in-plane wall response, out-of-plane wall response, and the response of wall corners. The changes in the dynamic structural characteristics and the force transfer mechanism due to seismic retrofit applications comparing to the as-built structure were the primary goals to be evaluated. The basic dynamic parameters such as natural frequencies and mode shapes were also determined using non-destructive forced vibration tests. The results from the tests conducted are expressed in terms of a frequency-dependent ratio between the output response and the input excitation, which is typically known as a frequency response function (FRF).

2. AS-BUILT AND RETROFITTED UNREINFORCED MASONRY HOUSE

The unreinforced masonry house having a 4 m × 4 m plan was constructed using a skilled bricklayer as shown in Figure 1. The north wall was a single leaf 110 mm thick solid brick wall with a 1.91 m height, whereas the south, west and east walls were double leaf 230 mm thick brick walls with opening(s) and a 2.2 m height (refer to Figures 2 and 3). The south wall was constructed with a door and window opening, while the west and east walls had a window opening only. All unreinforced masonry walls were built using recycled solid clay brick units where the dimension of each unit was 110 mm × 80 mm × 230 mm. For the double leaf thick walls, they were laid in common bond with a header course at every fourth course using a mortar mix proportion of 1:2:9 (cement:lime:sand). These construction methods and materials were used to replicate typical unreinforced masonry construction in New Zealand (Abdul Karim et al. 2009a).

The unreinforced masonry house had a timber diaphragm constructed with six 45 mm × 140 mm timber joists, covered with straight 32 mm × 140 mm timber flooring planks. The joists were oriented parallel to the north and south walls at a 723 mm centre to centre spacing, and were supported by the interior leaf (pocket) of the east and west walls. 45 mm × 140 mm timber blockings were applied between joists at 1165 mm centre to centre spacing. In order to avoid the timber diaphragm being overly stiff, the timber flooring planks were placed in a staggered pattern as shown in Figure 4a. Two nails were provided at each plank's end. To examine the effect of a plywood diaphragm retrofit on the structural dynamic response of the unreinforced masonry house, 12 mm thick plywood boards were subsequently nailed on the top of the flooring as shown in Figure 4b. The structural dynamic response of unreinforced masonry house with wall-diaphragm connections was also investigated, so that, the effects of the connections on the structural dynamic response of the house can be identified. The type of wall-diaphragm connections incorporated was a through-bolt that having a visible bearing plate on the exterior face of the wall acting as wall anchorages and a steel angle bolted to the joists as diaphragm connections (see Figure 5). This connection type was commonly applied in New Zealand unreinforced masonry buildings as part of the seismic retrofitting techniques (Abdul Karim et al. 2009b).

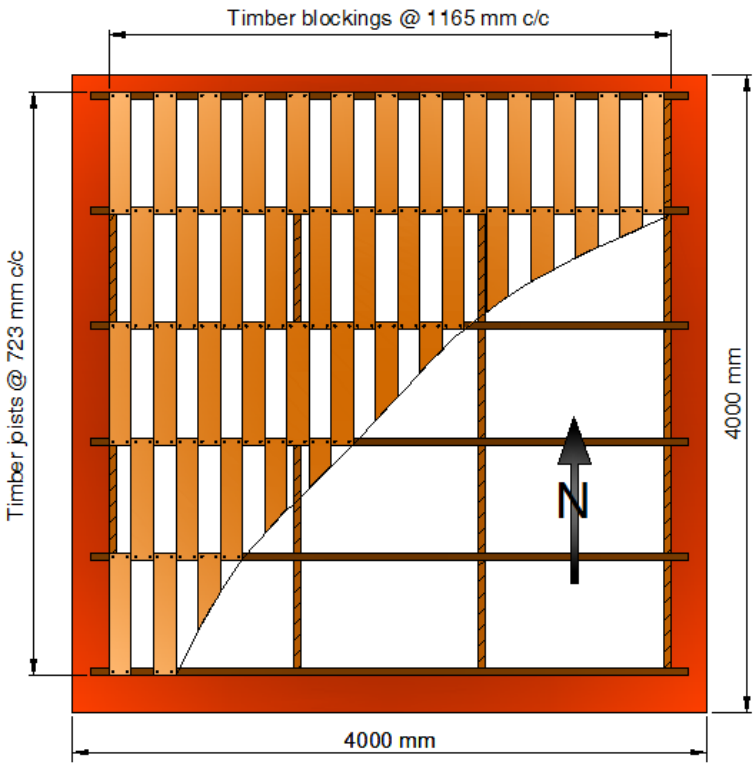
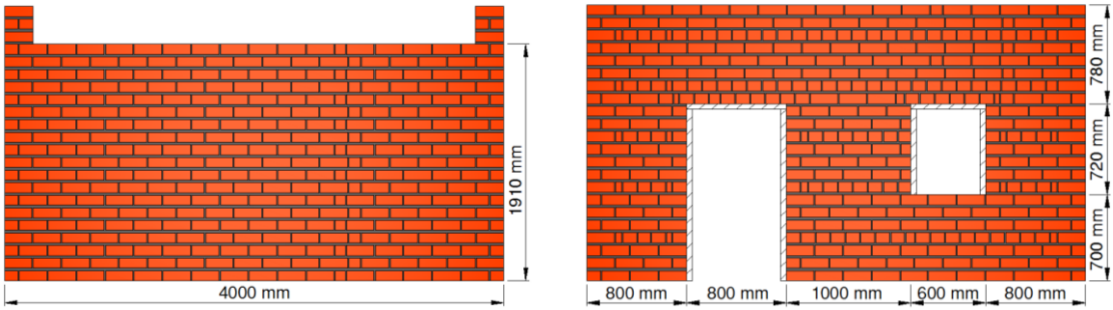
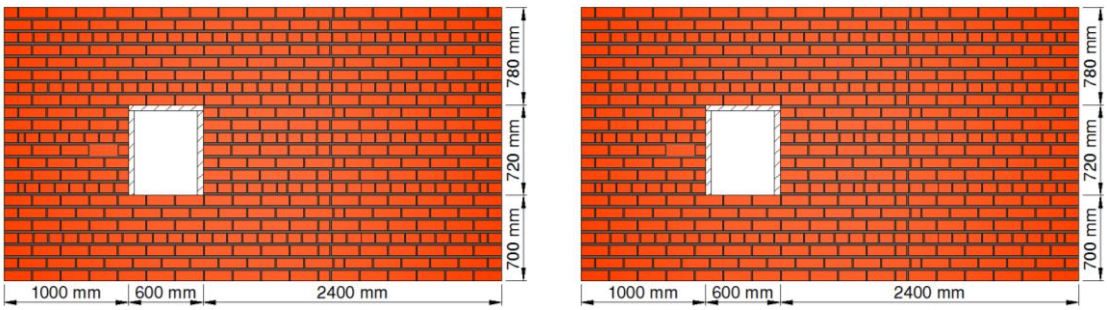


Figure 1: Plan view of the house.



(a) North wall (b) South wall

Figure 2: North and south elevations of the house.



(a) West wall (b) East wall

Figure 3: West and east elevations of the house.

(a) Flooring planks in staggered pattern (b) Flooring covered with plywood boards

Figure 4: North western view of the house showing as-built and retrofitted diaphragm.

(a) Wall anchorages (b) Diaphragm connection

Figure 5: Through-bolts type of connections implemented to the house.

3. NON-DESTRUCTIVE FORCED VIBRATION TESTS

Forced sinusoidal vibration tests were conducted using an APS Dynamics Model 400 electrodynamic mass shaker. The shaker was placed on the top of the timber diaphragm and operated in the horizontal direction to produce lateral vibration of the timber diaphragm. A small uniaxial accelerometer, attached to the arm of the shaker, was used to record the input excitation of the shaker. At each test conducted, a total of six accelerometers were used to measure the output response of the diaphragm and walls. Each accelerometer was mounted on a steel base plate that was equipped with a levelling bubble, thus, a proper alignment can be ascertained during the measurement of the output data. For data collections, all accelerometers were connected to a data acquisition box that equipped with 16-bit DAQ cards. The data acquisition box was then linked to a personal computer for recording of both input and output signals using a sample rate of 500 data per second. To control the shaker and data acquisition operations a MATLAB computer software (The MathWorks Inc. 2007) was utilised.

The experimental programme consisted of three main stages. The first stage of the testing was performed before any retrofitting done to the house (i.e. as-built), whereas the second stage was executed after retrofitting using a 12 mm thick plywood board, and the third stage was conducted after retrofitting using a through-bolt anchor connection applied between the walls and diaphragm members. Before conducting the actual testing, a trial non-destructive forced vibration test was conducted for each stage to determine the frequency response at the first translational mode of the structure, so that, a targeted frequency range of interest was established for actual testing. Details of the trial tests for all stages are summarised in Table 1. The selection of the frequency range of stage 1 was based on a finite element model prepared, which the details have been published elsewhere (Ingham et al. 2008, Abdul Karim et al. 2008). The shaker excitation was oriented in two different directions, which were north-south

(NS) and east-west (EW), to investigate the effect of loading direction on the force transfer mechanism in the unreinforced masonry building.

Table 1: Summary of trial forced vibration tests.

Stage	Shaker excitation	Frequency range (Hz)	Frequency step (Hz)	Mode frequency (Hz)	Targeted frequency range (Hz)
1	NS	10-20	1	12	10.0-15.0
	EW	10-20	1	13	
2	NS	5-50	1	16	14.5-19.5
	EW	5-50	1	16	
3	NS	5-50	1	19	17.0-22.00
	EW	5-50	1	18	

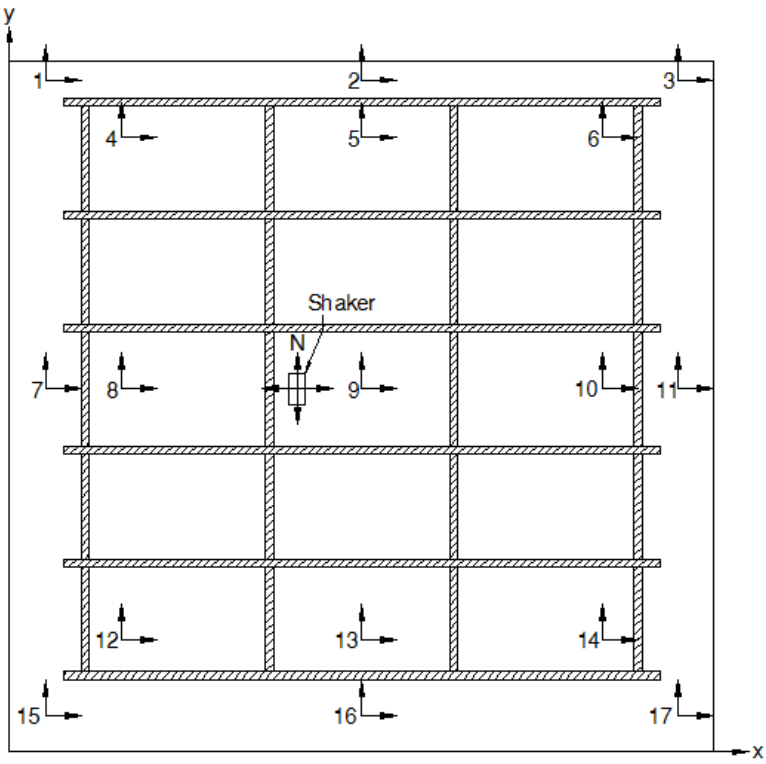


Figure 6: Location and direction of accelerometers and shaker.

In order to obtain sufficiently accurate vibration properties of the diaphragm and walls for the actual non-destructive forced vibration tests, a total of seventeen test points were established to measure the diaphragm and wall accelerations as shown in Figure 6. Similar to the trial tests, the shaker excitation was also oriented in two different directions when conducting the actual tests. The measurement of output signals of each test point was done in both *x*-axis and *y*-axis for each excitation direction, so that, the translational changes of the structure mode shape can be plotted. A larger number of test points results in a more refined mode shape. Each test in stage 1 was conducted by issuing a stepped sine input motion command to the shaker, which the frequency was gradually increased from 10 Hz to 15 Hz with a step increment of 0.1 Hz. Zero amplitude phase or time delay of 5 seconds was included before each increment of the frequency to permit the vibration of the structure to become stationary. The zero amplitude phases were also included to allow any structural response that was related to a particular frequency to dissipate before the new frequency was applied (Wilson et al. 2008). Similar procedures were used in conducting all tests in Stage 2 and Stage 3 except

the targeted frequency ranges applied were 14.5-19.5 Hz and 17.0-22.0 Hz, respectively. Details of the actual non-destructive forced vibration tests for all three stages are summarised in Table 2.

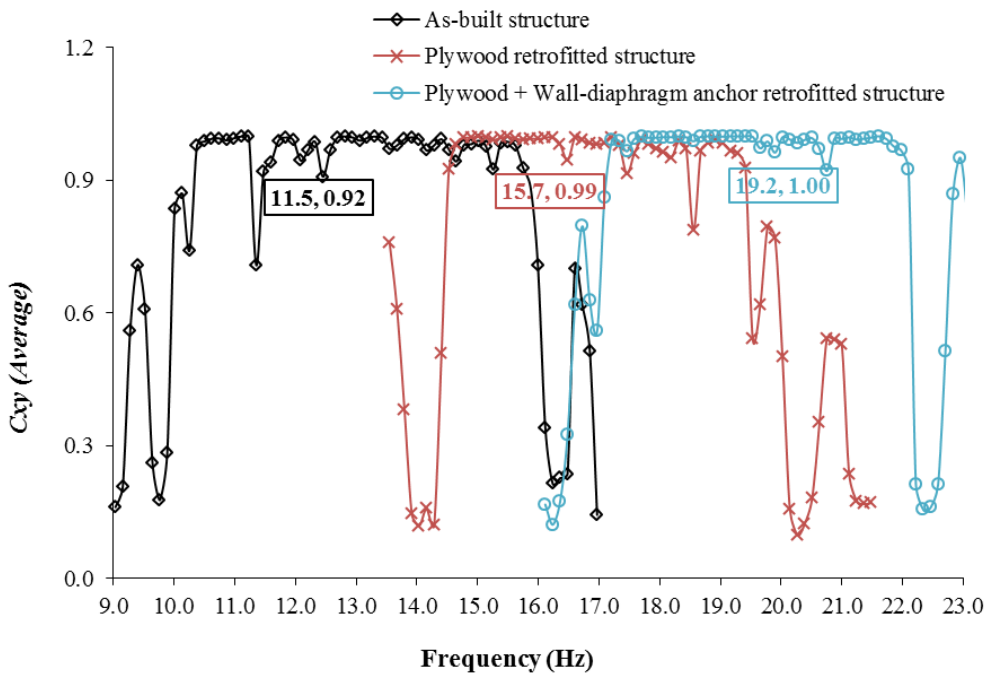
Table 2: Details of the actual non-destructive forced vibration tests.

Stage	Shaker excitation	Number of test	Test points		Freq. range (Hz)	Freq. step (Hz)	Excitation (s)	Time delay (s)
			x-axis	*y*-axis				
1	NS	1	1 to 6	–	10.0-15.0	0.1	15	5
		2	7 to 11	–				
		3	12 to 17	–				
		4	–	1 to 6				
		5	–	7 to 11				
		6	–	12 to 17				
	EW	7	1 to 6	–	10.0-15.0	0.1	15	5
		8	7 to 11	–				
		9	12 to 17	–				
		10	–	1 to 6				
		11	–	7 to 11				
		12	–	12 to 17				
2	NS	1	1 to 6	–	14.5-19.5	0.1	15	5
		2	7 to 11	–				
		3	12 to 17	–				
		4	–	1 to 6				
		5	–	7 to 11				
		6	–	12 to 17				
	EW	7	1 to 6	–	14.5-19.5	0.1	15	5
		8	7 to 11	–				
		9	12 to 17	–				
		10	–	1 to 6				
		11	–	7 to 11				
		12	–	12 to 17				
3	NS	1	1 to 6	–	17.0-22.0	0.1	15	5
		2	7 to 11	–				
		3	12 to 17	–				
		4	–	1 to 6				
		5	–	7 to 11				
		6	–	12 to 17				
	EW	7	1 to 6	–	17.0-22.0	0.1	15	5
		8	7 to 11	–				
		9	12 to 17	–				
		10	–	1 to 6				
		11	–	7 to 11				
		12	–	12 to 17				

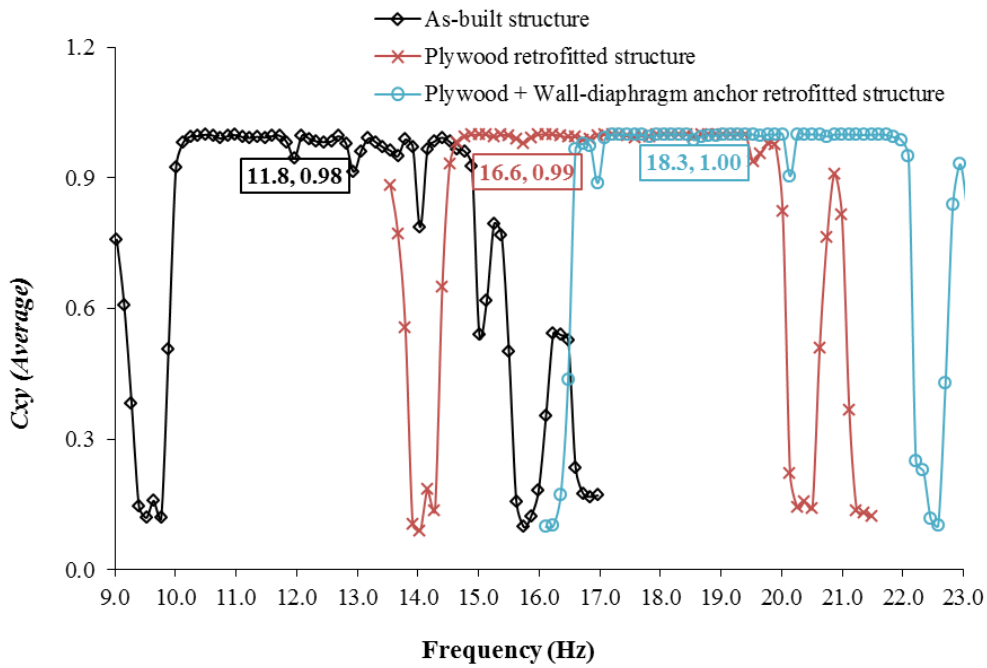
4. RESULTS AND DISCUSSION

Before analysing the data, the average coherence (*Cxy*) values between the output and the input signals for all test points was calculated in order to evaluate how well the output signal corresponds to the input signal at each frequency measured. From Figure 7, the average *Cxy* values were over 90% at every mode frequency measured. This indicates that the measured

output signals were not significantly contaminated with any random background noise or other sources of vibrations. In other words, the output signal collected from each accelerometer used is considerably corresponded to the input signal, thus, a good quality of output data is achieved.



(a) NS excitation

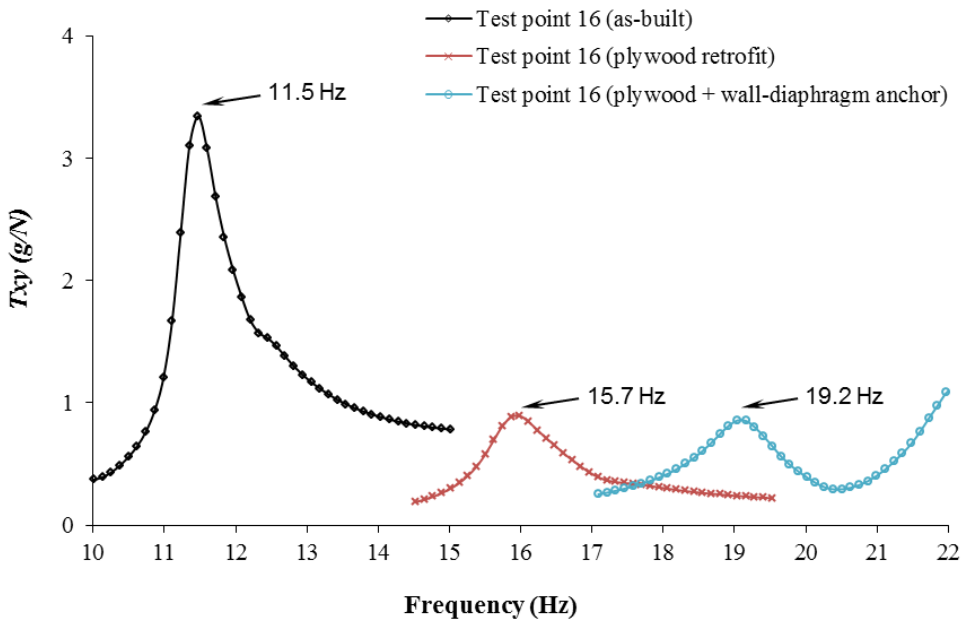


(b) EW excitation

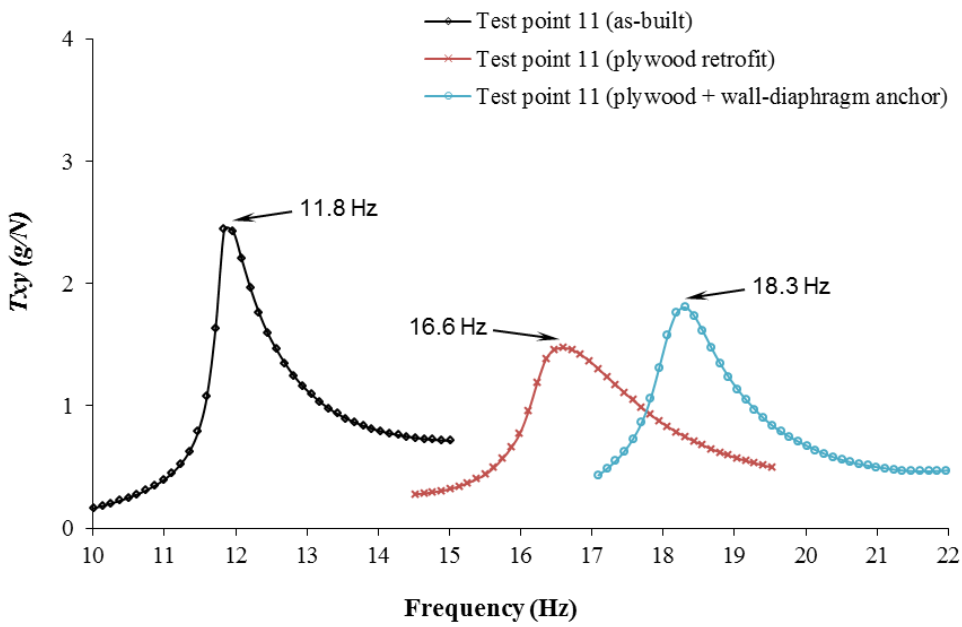
Figure 7: Typical coherence function curves

The data collected from the actual forced vibration tests were then analysed and expressed in terms of the frequency-dependent ratio between the output response and the input excitation. The relationship between the input (x) and output (y) was modelled using a linear, time-

invariant transfer function (*Txy*), which is the quotient of the cross power spectral density (*Pyx*) of *x* and *y* and the power spectral density (*Pxx*) of *x* (The MathWorks Inc. 2007). From the *Txy* curves (see Figure 8), the natural frequencies corresponded to the peak *Txy* amplitude can be determined. The natural frequencies were shifted from 11.5 Hz to 15.7 Hz to 19.2 Hz and from 11.8 Hz to 16.6 Hz to 18.3 Hz for NS and EW excitations, respectively. All transfer function curves shown in Figure 8 are representing the frequency response at the first translational mode of the structure.



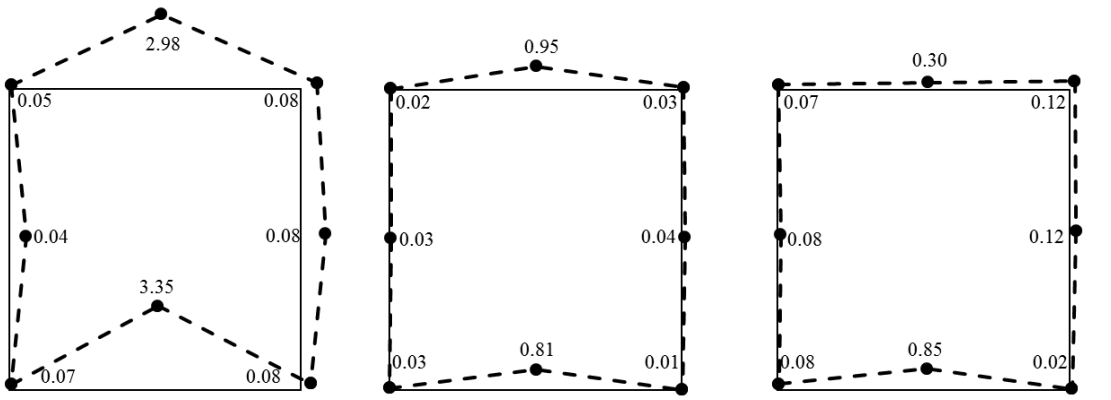
(a) NS excitation



(b) EW excitation

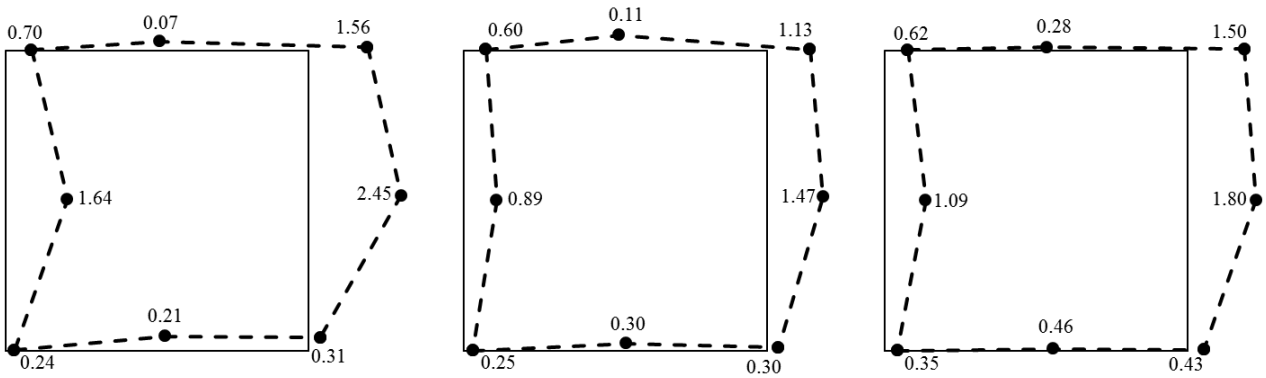
Figure 8: Typical transfer function curves

By identifying the peak *Txy* amplitudes at each test point, the translational mode shapes for the NS and EW excitations can also be established as illustrated in Figures 9 and 10. The values shown in both Figures 9 and 10 are the peak amplitudes for *y*-axis and *x*-axis at the natural frequency of the structure for test points 1, 2, 3, 7, 11, 15, 16 and 17 for NS and EW excitations, respectively. Only peak amplitude values of x-axis are shown for WE excitation (Figure 10), as the y-axis values are very minimal whereas this is vice versa for NS excitation (Figure 9). For the as-built structure that excites in NS direction, the amplitudes of out-of-plane walls (i.e. test points 2 and 16) are 37 times (min) and 84 times (max) higher than the amplitudes of in-plane walls (i.e. test points 7 and 11). This clearly indicates that, due to the non-existence of shear connections between in-plane walls and diaphragm, the as-built structure had a deficiency in transferring forces from the diaphragm to the in-plane walls acting as shear or resisting walls, even when the joists were inserted into a pocket of the interior leaf of the in-plane walls. The orientation of the joists parallel to the north and south walls may also contribute to the higher amplitude values in the out-of-plane walls as the joists are more flexible to bending when they were stimulated in NS excitations. Consequently, it causes the pounding of the diaphragm to the walls. The effect of different joist orientations to the walls amplitudes can be seen when the excitation was applied in the EW direction as the out-of-plane wall amplitudes (i.e. test points 7 and 11) are only higher than the in-plane wall amplitudes (i.e. test points 2 and 16) from 8 times to 35 times of the amplitude values.



(a) As-built (b) Plywood retrofitted (c) Plywood + Anchor retrofitted

Figure 9: Mode shapes of the unreinforced masonry walls for NS excitation



(a) As-built (b) Plywood retrofitted (c) Plywood + Anchor retrofitted

Figure 10: Mode shapes of the unreinforced masonry walls for EW excitation

By applying the plywood-retrofit on the diaphragm, the responses of the out-of-plane walls and wall corners were clearly identified. The amplitudes of the out-of-plane walls and wall corners were significantly reduced for the structure with plywood retrofitted diaphragm when

compared to the as-built structure. The reductions of the out-of-plane wall amplitudes were in the range of 3-4 times and 1.7-1.8 times for NS and EW excitations, respectively. The wall corner amplitudes reductions were found to be in the range of 2.3-8.0 times for NS and 1.0-1.4 times for EW. These results indicate that the plywood retrofitted diaphragm improved the force transfer mechanism compared to the non-retrofitted diaphragm in the structure. As a consequence, the bending failure of the out-of-plane walls, the pounding between the diaphragm and the unreinforced masonry walls, and the failure of wall corners can be significantly controlled.

In a comparison between the plywood retrofitted and plywood + anchor retrofitted structures for NS excitation, one can see that the amplitude of in-plane walls was increased to 2.8 times in average. Meanwhile, for EW excitation, the average amplitude increment was about twice. From the results obtained, the installation of a through-bolt anchor type between the unreinforced masonry walls and timber diaphragm shows an improvement in transferring the force to the in-plane walls which can avoid the walls to collapse as they are better in resisting the force in-plane. Although, the anchor installations showing an increment to the out-of-plane wall amplitudes, the placement of the anchors should be considered as the earthquake loadings can be struck from any directions with respect to the building orientations.

In Figure 10, one can see that the higher peak amplitude values of in-plane walls are mainly affected by the wall thickness but not the opening of the wall whereas the north wall was constructed as a single leaf with no opening compared to the double leaf south wall with door and window openings. This is also can be seen in the NS excitation (Figure 9) as both west and east walls showing low peak amplitude values even with windows openings.

The reader must be aware that all results presented may be significantly affected for larger scale of masonry buildings as they are having taller storey heights and wider timber floor diaphragms, which causing both walls and diaphragms to be more flexible with higher amplitude values.

5. CONCLUSIONS

From the non-destructive forced vibration tests conducted, the natural frequencies and mode shapes of the first translational mode of the unreinforced masonry structures, either as-built or retrofitted, were evaluated. The structural dynamic responses of walls, diaphragm and building corners due to the implementation of the retrofitting techniques were also identified. The plywood and anchor applications to the as-built structure show distinctive advantages to the structure overall response. A better understanding on the force transfer mechanism of the unreinforced masonry structures was also achieved.

6. ACKNOWLEDGEMENTS

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