Seismic Resistance of Brick Veneer Under Face Loading

Stuart J. Thurston[1] and Graeme J. Beattie[2]

1 Senior Engineer. BRANZ Ltd. Email: Stuart.Thurston@branz.co.nz

2 Principal Engineer. BRANZ Ltd. Email: Graeme.Beattie@branz.co.nz

Abstract

Recent full-scale cyclic and shake tests by the authors have shown that modern single and two-storey brick veneer New Zealand houses will perform very well under in-plane testing and can be relied upon to carry a significant portion of the seismic design load. After a design level earthquake cracking is expected to be almost imperceptible and even after a maximum considered event little or no veneer collapse is expected. Cracks should be repairable simply by raking out mortar along crack lines and repointing.

This paper describes out-of-plane shake tests on seven 2.4 m wide x 2.23 m high brick veneer walls fixed to 2.42 m high light timber framing (LTF) using sinusoidal loading over a frequency range of 0.5 to 10 Hz. The walls had been pre-racked in-plane to deformations implied by AS/NZS 2699.1 *Built-in Components for Masonry Construction – Part 1: Wall Ties*. Acceleration spectra determined from measured table accelerations are compared with design action coefficients for New Zealand single and two-storey brick veneer buildings and by this means it is shown that good seismic performance can be expected in service. Variables considered included tie-type, full encapsulation versus dry bedding and the effect of openings in the veneer.

Keywords: seismic, earthquake, brick, veneer, face loadings, shake table.

1. INTRODUCTION

Historically brick veneer houses have not performed well in earthquakes. Much blame for the lack of confidence in brick veneer is due to failures in the 1931 Napier earthquake, the 1968 Inangahua earthquake and the 1987 Edgecombe earthquake. However, recent tests at BRANZ (Thurston and Beattie, 2009a) have shown that modern single-storey and two-storey construction performed excellently at well past the design level displacements. This was attributed to the use of better brick-ties which are screwed to studs and the use of cored bricks (i.e. with internal holes). However, these tests effectively only tested the veneer under in-plane loading. The current study investigates whether the veneer will continue to perform well under out-of-plane loading, having first been subjected to in-plane loading.

Brick veneer inertial face loads are transmitted to the LTF by brick ties. Failure can occur by fracture of the ties, pullout of the tie fixing from the timber framing or brick mortar, failure of the timber framing in flexure or by separation of the timber stud-to-plate joint. Shedding of masonry units will usually occur as a consequence of one of the above failure mechanisms. Cracking of the veneer under the design-level earthquake does not constitute failure, as this would imply a more severe requirement than demanded of engineered structures. Repair of a competent, but cracked, veneer after an earthquake should merely be a matter of repointing.

Thurston and Beattie (2009b) conducted a literature survey of in-plane and out-of-plane tests on brick veneer. More detailed information on the testing described in this paper is given in this report. If brick ties are nailed to timber framing, stud vibration can loosen ties in partially set mortar due to the nailing action. Screwing ties to studs results in far greater resistances to face load as both the fixing to timber and mortar-to-tie bond are stronger. Partially loose ties can result in impulse loading in an earthquake as the tie forces increase. The veneer can resist intense shaking if ties are screwed when other failure mechanisms (e.g. stud fracture and failure of stud-to-plate joints) become the weakest links.

Thurston and Beattie (2009b) showed that the seismic design action coefficient on brick veneer as prescribed by NZS 1170.5 *Structural Design Actions. Part 5: Earthquake actions – New Zealand*. (SNZ, 2004) is 1.2 for single-storey buildings and 3.36 for the upper storey of two-storey buildings. The design level horizontal force is the veneer self-weight multiplied by the design action coefficient.

2. SPECIMEN CONSTRUCTION

Details of the seven walls tested are summarised in Tables 1 and 2. The veneer was 2.4 m wide and consisted of 26 courses of brick resulting in a veneer height of 2.23 m. The ties were at the 2nd and 24th mortar joint and every 4th joint between. The walls sometimes included 0.6 m wide windows or doors as shown in Table 2. The wall framing was 1.8 m wide and 2.42 m high and consisted of 90 x 45 mm MSG 8 radiata pine studs at 600 mm centres. The ties were dry-bedded in the first four walls but in the remaining three walls the ties were fully embedded in the mortar.

Table 1. Details of walls tested

Wall Label	Veneer Shape	Tie Type	Framing Type	Tie Embedment	Top support
W1	2	Eagle	Timber	Dry bedded	Full
W2	1	Eagle	Timber	Dry bedded	Full
W3	1	Eagle	Timber	Dry bedded	Full
W4	3	Eagle	Timber	Dry bedded	Full
W5	1	Eagle	Timber	Full embedment	Full
W6	1	Eagle	Timber	Full embedment	Flexible
W7	1	Mitec	Timber	Full embedment	Full

Table 2. Shapes of brick veneer tested

Veneer Shape	Description	
1	Rectangular, 2.4 m wide x 2.23 m high.	
2	Contains 300 mm wide edge windows with no lintel	
3	Contains a 600 mm wide central door with a 3-brick high lintel	

3. BRICKS, BRICK-TIES AND MORTAR

The bricks used in the BRANZ tests had dimensions 230 long x 76 x 70 mm wide. They had five vertical ‘core’ holes of cross-section 32 x 23 mm for the full brick depth, mainly to lighten the brick and aid the firing process. When the bricks are laid, the holes partially fill with mortar, thus effectively forming mortar dowels, which greatly enhances the horizontal shear strength between bricks.

The bricks were laid by tradesmen using pre-bagged dry mortar. The mortar had a measured 28 day crushing strength of 7.3 MPa and the joints were concave tooled and burnished after the initial stiffening had occurred.

Commonly available 85 mm long, hot-dipped galvanised, brick-ties (either Eagle or Mitec brand as shown in Table 1) were spaced 340 mm vertically and 600 mm horizontally. Ties were secured to the face of the timber studs using 35 mm long galvanised Tek screws.

4. IN-PLANE TESTING PRIOR TO THE OUT-OF-PLANE TESTS

The walls were slow cyclically racked in the in-plane direction (Figure 1) before performing shake tests in the out-of-plane direction. The wall framing was fixed to preclude stud uplift. The wall frame was lined with plasterboard for the in-plane tests but this was removed for the out-of-plane shaking tests as it provided no contribution to the strength and little inertial mass.

For the in-plane tests, an actuator applied four cycles of ±24 mm top plate displacement to each framed wall with the brick veneer precluded from rocking, and then four cycles to ±36 mm with the rocking restraint removed. The veneer walls cracked at opening corners and at the base of the wall.

Figure 1. Test set-up for in-plane racking tests

Figure 2. Test set-up for out-of-plane shake table tests

5. OUT-OF-PLANE TESTING

5.1 Test set-up

The bottoms of the specimens were bolted to the shake table as shown in Figure 2. The top of the wall framing was constrained to move in concert with the base of the wall. Thus, the accelerations imposed on the table were also (approximately) the same as imposed at the top of the framing. This simulated the situation where the side walls and ceiling/roof diaphragm of a house are extremely rigid. To check whether side wall and diaphragm flexibility would result in a more severe loading on the veneer, in one test (Wall W6) a flexible steel cantilever system was used between the top plate and adjacent support frame. This resulted in a maximum of 7 mm relative movement between the two.

5.2 Test regime

Walls were subjected to four cycles of sinusoidal displacement at frequencies commencing at 10 Hz and then reducing to 1 Hz generally in steps of 2 Hz, but smaller steps were used on either side of the measured wall natural frequency. All shaking had an additional slow lead-in and tail-out cycle to avoid any shock loading. Accelerations were measured at floor level and mid-height and the top of both the LTF and veneer.

The maximum shaking acceleration in each stage of loading was set to either a constant value or, if the actuator could not achieve this level of acceleration, then the shaking was set to the maximum acceleration the table could provide. In the early stages of each test the motion at all frequencies was set at the pre-determined table acceleration level. At higher target acceleration levels the achieved accelerations at the lower frequencies was limited by the table capacity.

6. NATURAL FREQUENCY MEASUREMENTS

Priestley et al (1979) developed a formula for calculating the natural frequency of veneer panels. They found that the natural frequency of their test walls commenced at close to the theoretical uncracked natural frequency and then migrated to close to the theoretical cracked natural frequency as the tests progressed. Inserting the construction dimensions used in the BRANZ tests into their formula results in an uncracked natural frequency of 9.1 Hz and a cracked natural frequency of 4.5 Hz for a wall without openings.

6.1 Free vibration tests

The natural frequency of each test wall was determined by cycling the shake table to large displacements and then abruptly bringing the motion to a stop. This created a pulse motion on the wall which quickly settled into a decaying free vibration allowing the wall natural frequency and damping to be calculated.

An example of the differential displacement between veneer and framing at the top of the veneer recorded during such free vibration motion is shown in Figure 3. Almost identical natural frequencies were obtained from the measured veneer mid-height acceleration and also the differential displacement between veneer and framing at the veneer mid-height. However, these latter plots were less smooth, showing the influence of other contributing frequencies.

The measured natural frequency of all test walls at the start of the out-of-plane testing is shown in the second column of Table 3. As the walls had been ‘pre-cracked’ during the in-plane testing it is not surprising that their natural frequency was close to the theoretical cracked frequency. After further testing, but before many ties had slipped within the mortar, the measured wall natural frequency showed less than 0.5 Hz reduction and the damping increased by less than 0.4%. However, in the shaking just prior to failure, when many ties had slipped within the mortar, the damping approximately doubled and the natural frequency dropped from the initial value by as much as 1.3 Hz.

The measured wall natural frequency varied between 5.7 to 6.3 Hz for all walls except for the wall with a flexible top support (Wall W6) which was 5.3 Hz.

6.2 Resonant frequency tests

A typical plot of the peak differential displacement between the LTF wall and veneer versus the shaking frequency is shown in Figure 4. It can be seen that the resonant frequency slightly reduces as shaking intensity increases.

Table 3 shows that the measured resonant frequencies are generally slightly lower than the natural frequencies determined from the initial free vibration tests. The resonant frequency measured at the top of the veneer reduced to low values (2-4 Hz) when the top portion began rocking independently.

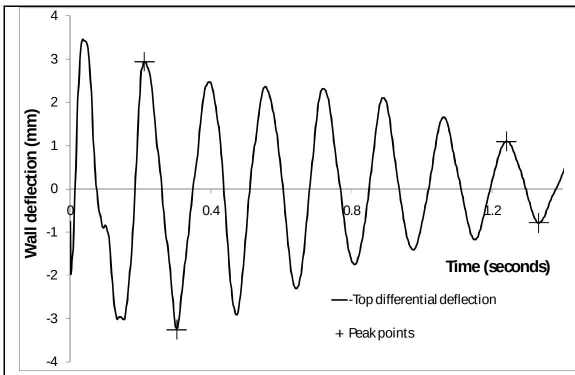

Figure 3. Free vibration wall response

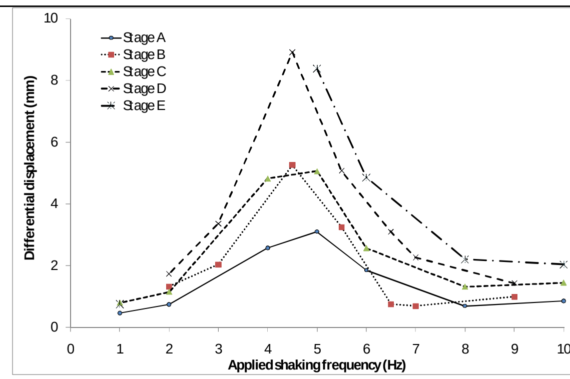

Figure 4. Wall mid-height veneer-to-framing displacement versus frequency

7. OUT-OF-PLANE TEST RESULTS

The third column of Table 4 describes the wall failure mechanisms and Figure 5 shows photographs of veneer shedding. In five of the seven walls the top portion of bricks toppled in a cantilever action. Subsequent shaking resulted in further layers falling from the top. In the other two walls (Wall W1 and W7) the veneer collapse occurred in the body of the veneer.

In all instances veneer shedding was due to tie failure. Generally this was preceded by full tie pullout from the mortar, although occasionally the ties partially pulled out under tension, jammed and then buckled under subsequent compression load and then finally ruptured. In two instances (Wall W5 and W7) stud rupture and shear failure of stud-to-top plate joints occurred before the veneer collapsed. These LTF failures were repaired and the testing continued until veneer shedding occurred.

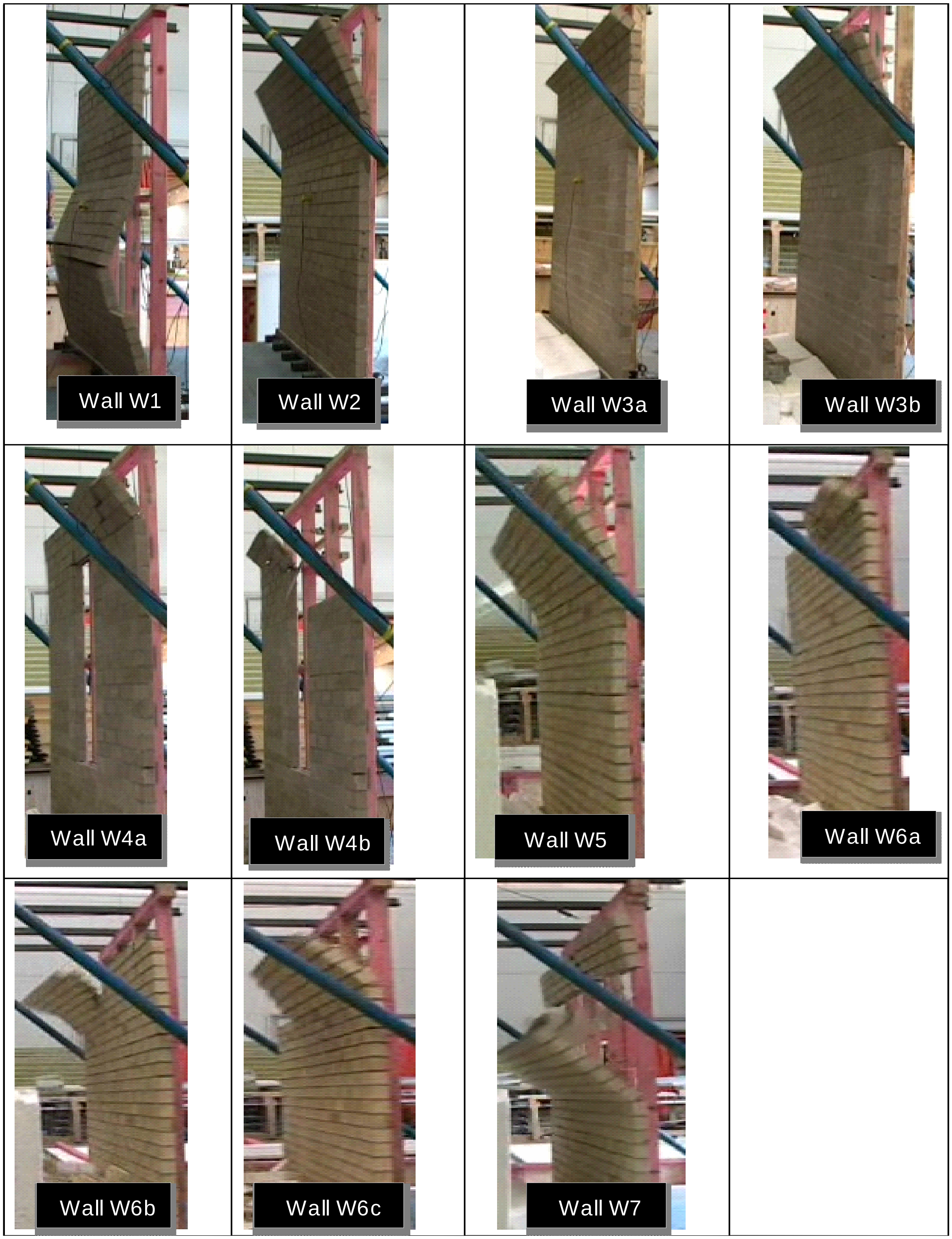

Figure 5. Photographs of veneer collapse mechanisms for each test

8. RESPONSE SPECTRA

Although the sinusoidal accelerations applied in the tests are vastly different to the signal from real earthquakes it is possible to correlate results obtained from these tests with expected performance in an earthquake. This is because the panels in any test behave in an essentially elastic manner (i.e. natural frequency and damping remain constant). Thus, if these values are known, then acceleration spectra can be computed from the measured table accelerations.

The applied table displacements were a smooth sinusoidal record as shown in Figure 6. However, due to friction effects the table accelerations were a rougher signal. The acceleration records were used to compute acceleration spectra and an envelope of such spectra from tests before that causing failure was computed for each wall. The spectra presented herein were derived for 5% damping as this is what was assumed when the NZS 1170.5 (SNZ, 2004) design spectra were derived.

At low frequencies the maximum acceleration able to be applied was limited by the speed of the actuator resulting in the spectra plots initially rising almost linearly with frequency. At mid-range frequencies, the table displacement was selected to give an approximately horizontal slope in the 5–8 Hz range for each stage of test shown in Figure 7 and 8, although due to table feedback there was actually some increase in the spectra near the wall natural frequency. Between 8–10 Hz the spectra also tended to increase. This was again attributed to feedback from the table and end support frame. As this frequency range was not critical to the performance of the test walls, this feature is considered to be of little consequence.

The spectral envelope calculated for walls W1 to W4 (i.e. the walls where the ties were dry-bedded) is shown in Figure 7. Also shown in this figure are the design action coefficients discussed in Section 1. The spectra just prior to failure of the first test wall (Wall W1) were far higher than the design values for frequencies above 5 Hz.

Based on the good results for Wall W1, the shaking for Wall W2 commenced at a high level. However, Wall W2 failed earlier than expected and was not tested at frequencies less than 7Hz. The mortar for this particular wall appeared to be poorly bonded and was laid during a period of hot dry weather. It is thought that the bricklayer had used mortar which had become too dry for the conditions.

Before the out-of-plane shaking had commenced on Wall W3, it was noted that a crack had formed in the second mortar joint from the top (where the ties were embedded), possibly from the in-plane cycling. The top two courses rocked from the first stage of loading and toppled during the first testing at 3Hz (see the photograph for Wall W3a in Figure 5). The test continued on this wall until a second veneer collapse occurred (Wall W3b in Figure 5). The spectra before this second collapse were far higher than that for the first collapse.

Wall W4 incorporated a large window opening, with four courses of brick above the opening. The cracks which formed from the in-plane cycling on mortar joints corresponding to the top and bottom of the window passed through the tie locations. At the top of the window, ties were accidently placed under the steel angle lintel seating which meant they were not bonded. This resulted in a relatively early failure of the top four courses (falling at 2Hz shaking). At shaking levels only slightly greater, other layers of veneer peeled off the top (see Figure 5).

Table 3. Measured wall natural frequency

Wall Label	Frequency from initial free vibration tests (Hz)	Wall resonance frequency determined from monitored response versus forcing frequency at the following gauges: Veneer mid-height accelerations (Hz)	Differential deflection at wall mid-height (Hz)	Differential deflection at top of veneer (Hz)	Damping from initial free vibration tests
W1	6.3	5.5 to 4.5	5 to 4.5	5.5 to 2	2.60%
W2	5.9	-	-	-	2.90%
W3	5.9	5.5 to 5	5.5 to 4	5 to ?	5.60%
W4	6.7	6 to 6	6 to 6	6 to 2	5.20%
W5	5.7	5 to 4.5	5 to 4.5	4.5 to 4	2.60%
W6	5.3	5 to 4.5	5 to 4.5	4 to ?	3.70%
W7	5.8	5.5 to 5	5 to 4	5 to 3	3.40%

Table 4. Test results

Wall Label	Stage	Failure description	5 Hz Spectra (g)	Table peak acceleration at 5 Hz prior to failure (g)	Ratio	Amplification at resonance = ratio of mid height brick to table accel.
W1	1	Bottom 16 layers fell.	6.4	1.04	6.17	6.76
W2	1	Top 7 layers fell.	-	0.47	-	-
W3	1	Top 2 layers fell	3.3	0.74	4.46	5.72
	2	Next 10 layers fell.	6.5	1.09	5.94	
W4	1	Top 4 layers fell.	3.3	0.56	5.94	4.21
W5	1	One middle stud cracked. This was repaired.	3.45	0.67	5.17	5.84
	2	Top plate-stud joint failed studs. Fillets installed.	5.41	0.92	5.89	
	3	Wall fallen from level 18 and above.	5.5	0.96	5.72	
W6	1	Top two layers fell along entire wall plus 5 layers from west end.	5.33	0.85	6.25	6.41
W7	1	Top plate loose on studs. Fillets installed.	3.5	0.61	5.74	
	2	One end and one middle stud cracked. These were repaired.	3.5	0.85	4.12	
	3	Middle section (levels 12 to 22 fell). Top 4 layers left hanging.	5.64	1.04	5.42	7.03

The spectral envelope calculated for walls W5 to W7 (i.e. the walls where the ties were fully embedded) is shown in Figure 8. Walls W5 and W7 were of similar construction except Wall W7 used Mitec rather than Eagle ties. Both performed well.

Wall W6 was similar to Wall W5 but it had a flexible top connection that moved ±7 mm near peak loading. This top flexibility did not appear to affect the wall performance. The result for Wall W1 from Figure 7 is repeated in Figure 8 for comparison and shows that dry bedding ties can be as effective as full bedding.

Figure 9 gives the spectra when timber framing failures occurred in Walls W5 and W7. These failures were well in excess of the design values for single-storey construction.

9. CONCLUSIONS

All seven walls resisted more than the design level single-storey earthquake loading for frequencies greater than 2Hz. Below 2 Hz the table actuator was unable to impose sufficient accelerations to fully test the walls. The body of the cracked veneer has a resonant frequency of approximately 5 Hz. The portion cantilevering above the top tie is most vulnerable at significantly lower frequencies. Standards currently limit the number of brick courses above the top tie to two. Closer tie spacing near the top of the wall, perhaps even using ties in perpends in the top course, would provide a better performance.

No veneer shedding or other failures occurred in any of the tested walls until well above the single-storey design levels. Many walls also resisted the shaking well beyond that corresponding to the two-storey design levels before veneer shedding occurred. Stud rupture and parting of the stud-to-top plate joints sometimes occurred near these shaking levels.

In-plane loading can induce cracks in the mortar. Such cracks can result in early tie pullout and thereby early veneer collapse. Poor mortar may produce a similar result.

10. ACKNOWLEDGEMENTS

This work was funded by the Building Research Levy. Elephant Plasterboard New Zealand donated the wall linings used in the testing described herein. Monier Brickmakers Ltd donated the bricks and Eagle Wire Products Ltd donated the brick-ties used in the testing.

11. REFERENCES

Priestley, M.J.N., Thorby, P.N., McLarin, M.W. and Bridgeman, D.O. (1979). ‘Dynamic Performance of Brick Masonry Veneer Panels’. *Bull. NZNSEE* 12 (4): 314-323.

SA/SNZ. (2000). AS/NZS 2699.1. *Built-in Components for Masonry Construction – Part 1: Wall Ties.* Standards Australia, Sydney, Australia.

SNZ. (2004). NZS 1170.5. *Structural Design Actions. Part 5: Earthquake actions – New Zealand*. Standards New Zealand, Wellington, New Zealand.

Thurston S.J. and Beattie G.J. (2009a). *Seismic Performance of New Zealand Two-storey Brick Veneer Houses.* Proceedings of the 2009 NZSEE Conference, Christchurch, New Zealand.

Thurston S.J. and Beattie G.J. (2009b). 'Seismic Performance of Brick Veneer under Face Loading'. BRANZ *Study Report 216*. BRANZ Ltd, Judgeford, New Zealand.

BRANZ Study Reports may be freely downloaded at www.branz.co.nz/cms_display.php

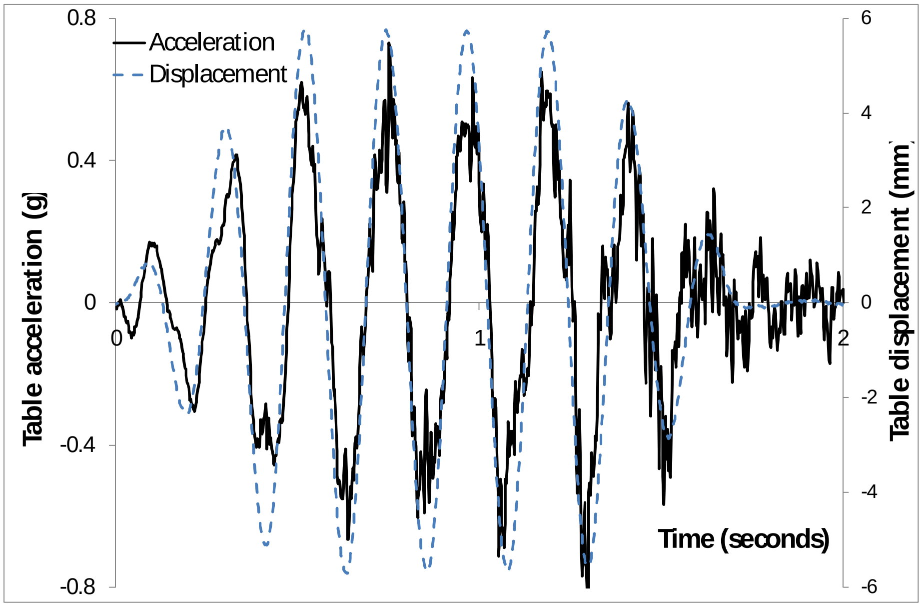

Figure 6. Typical plot of table displacement and acceleration in shake table tests

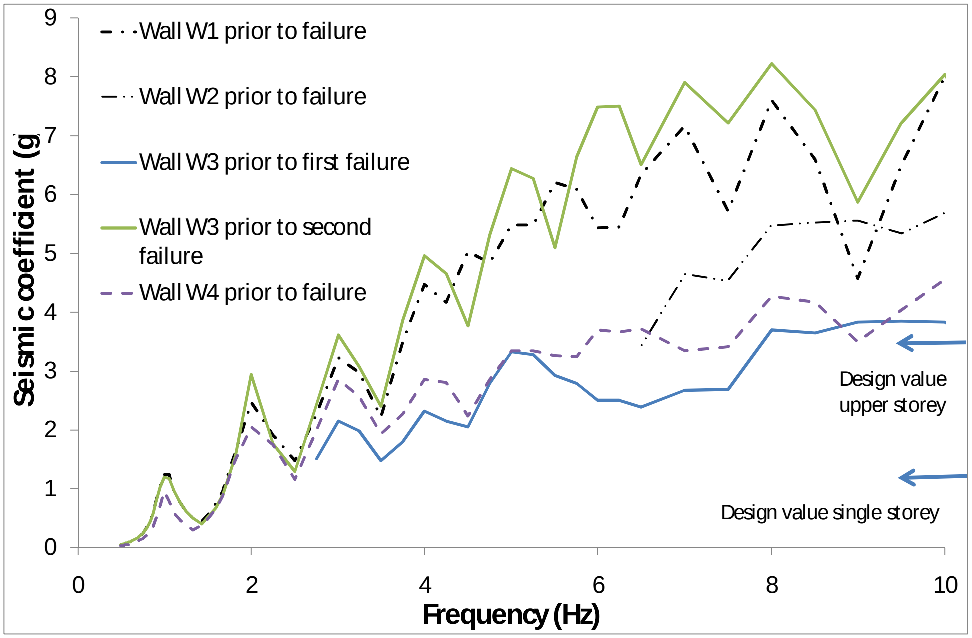

Figure 7. Spectra calculated from measured table accelerations of Walls W1 to W4

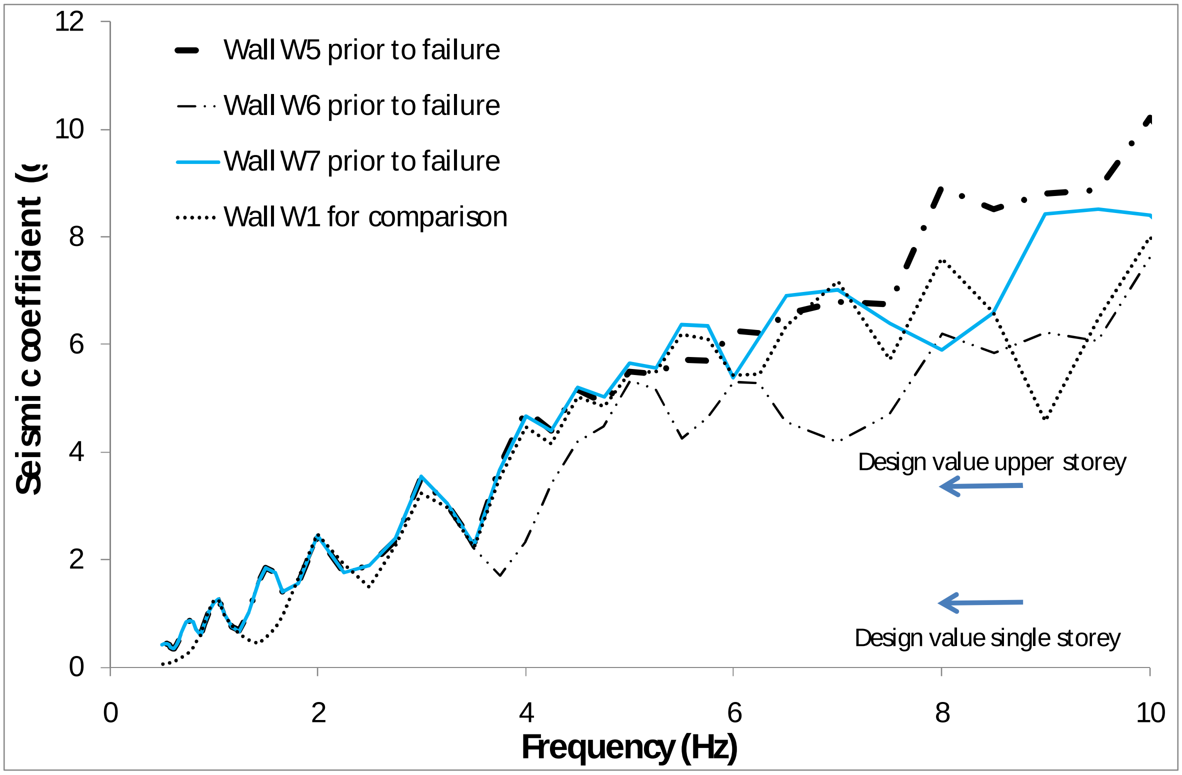

Figure 8. Spectra calculated from measured table accelerations of Walls W5 to W7

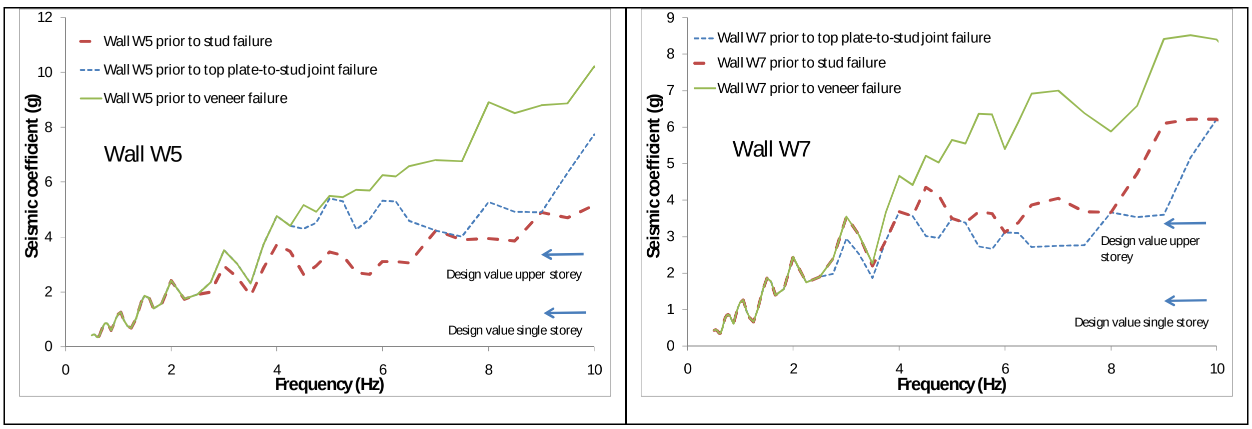

Figure 9. Spectra for Walls W5 and W7 for shaking prior to timber failure and veneer failure