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Seismic performance of brick veneer houses

Phase 3. Shake table tests on a concrete brick veneer room

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Preface

This is the fourth BRANZ investigation of a series looking into the seismic performance of brick veneer. The first two investigations were slow cyclic tests of full-scale veneer specimens where the veneer clad a rectangular room which had both window and door openings. The third investigation was a shake table test of a clay brick veneer clad room, with an inertial mass to simulate roof loads.

The quality of brick veneer construction has improved markedly in recent years, with requirements for the ties to be screw-fixed to the timber framing and with the advent of lighter bricks with vertical penetrations. It is considered that the veneer may no longer be just a driver, but rather that it may have some lateral load-resisting capability.

The complete study is intended to improve the understanding of brick veneer construction in earthquakes, in particular:

- To determine if brick veneer can be relied upon to carry most of the building seismic load or whether the building light timber framing construction should be designed to carry the entire load.
- To identify the level of seismic damage that can be expected in modern brick veneer construction.

It is intended to extend the scope of the study to investigate the performance of two-storey brick veneer.

Acknowledgments

This work was funded by the Building Research Levy. Elephant Plasterboard New Zealand donated the wall linings used in the testing in this report. Firth Industries Ltd provided the bricks at a discounted rate and Eagle Wire Products Ltd donated the brick ties used in the testing.

Note

This report is intended for the Department of Building Housing (DHB), standards committees, structural engineers, architects, designers, plasterboard and brick manufacturers and others researching this topic.

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BRANZ Study Report SR 191

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Reference

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Abstract

Historically brick veneer houses have not performed well in earthquakes. However, in Phase 1 and 2 of this project, modern construction using better brick ties which are screwed to studs, and the use of bricks with internal holes which allow mortar to penetrate and act as dowels, were shown to result in a better performance.

In this study, BRANZ performed sinusoidal shake table tests on a large concrete brick veneer room (including a ceiling). The room incorporated window and door openings. Brickwork cracking patterns were identified and rationalised. The magnitude of shaking was compared with that used on the clay brick veneer building tested in Phase 2 of this project.

The concrete brick veneer did not perform as well as the clay brick veneer. However, in shaking inducing light timber frame (LTF) deflections of approximately ±50 mm the veneer returned to close to its original position and veneer cracks largely closed, enabling repair by re-pointing. As the design LTF deflections are expected to be in the range ±16-36 mm in a design earthquake event, assuming no stiffening is provided by the concrete brick veneer, it was concluded that properly constructed concrete brick veneer could be relied upon to resist design earthquake events with low building deflections. The veneer would be easily repairable. However, severe damage and even collapse may occur under shaking several times the design event.

It is recommended that concrete brick veneer of the type tested not be relied upon for bracing in houses built to NZS 3604.

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1. INTRODUCTION

1.1 Background

Brick veneer is a popular form of cladding on New Zealand houses with 44% of new houses being clad with a veneer (BRANZ building database). Eighty four percent of veneer clad houses are clad with clay bricks while the remaining 16% are clad with concrete bricks.

Historically brick veneer has performed poorly in earthquakes. Cracking, particularly at building corners, window/door corners and partial collapse of the veneer have been common (Figure 1). However, recent improvements in the way brick veneer cladding is constructed is expected to result in improved performance and this study is intended to quantify this.

Figure 1. Examples of veneer damaged in the 1987 Edgecumbe earthquake

Traditionally, brick veneer has been assumed to be a driving element (mass) under earthquake loading and the timber framing has been the sole load-resisting wall element. NZS 3604 (SNZ 1999) has assumed that the brick veneer does not carry any in-plane seismic load and the brick veneer is a cladding element only which applies its inertial load to the timber-framed bracing walls during an earthquake.

There is a basic deflection incompatibility between the stiff veneer and the relatively flexible timber-framed wall which may lead to the development of damage in either in an earthquake under in-plane loading. It was generally considered that there is also a deflection incompatibility between the two orthogonal veneers at the corners of a building which could lead to significant vertical cracking and separation.

A complex inertial load transfer interaction is expected between the veneer and the timber frame because of the significant stiffness of the ties. The extent and effect of this load transfer was unknown before the commencement of the current testing at BRANZ. The existing NZS 3604 design philosophy assumes the veneer ties accommodate expected differential in-plane deflections between the framing and the veneer and the ties are expected to transfer all the inertial forces from the face-loaded veneer panels to the timber framing. However, in modern veneer construction, the pick-up of load by the veneer may indeed assist the overall performance of the building in an earthquake by reducing the displacement of the frame. On the other hand, it may also result in damage to the brick veneer.

Slow cyclic and dynamic tests on clay brick veneer specimens have been conducted in Phases 1 and 2 of this research programme (Thurston and Beattie 2008a & 2008b). Concrete bricks do not generally have vertical penetrations through the individual bricks, so it is not possible to achieve any mechanical interlock between bricks via the mortar courses. It was therefore considered essential that this type of construction be investigated to provide a comparison of performance between the clay and concrete veneers.

1.2 Reasons for dynamic testing

The principal advantage of undertaking dynamic over slow cyclic testing is that the inertial mass of the veneer, which is being mobilised in a real earthquake event, is also mobilised. The inertia effects are a critical input to the overall performance of the system.

In the tests in Phase 1 of this project (Thurston and Beattie 2008a), slow cyclic load displacements were imposed on room ceilings and the load transferred to the brick veneer was measured. There was no real time shaking performed. Thus, the testing carried out did not simulate the out-of-plane inertia forces on the brick veneer and thus the ability of the brick ties to transfer this load to the LTF construction was not tested.

The intersection of the face-loaded and the in-plane loaded veneer elements may perform differently under dynamic loading. It is possible that out-of-plane inertia forces may induce vertical cracking at the veneer corners which would separate the veneer sides from the ends. Phase 1 of this project showed that the integrity of this corner connection improves the sliding resistance of the side veneer panels. The rocking behaviour of the L-shaped corner veneer elements will be affected if cracking and separation occurs in the corners.

Phase 2 of the project tested the dynamic performance of the clay brick veneer, and produced very favourable results, with the veneer remaining well connected to the LTF and elements of multiple bricks functioning as a single element. Brick loss from the test specimen was effectively nil despite having been subjected to earthquake forces far in excess of current Code requirements.

1.3 New Zealand brick veneer construction

During the course of this research programme, a magnitude 6.8 earthquake struck the city of Gisborne, in Hawkes Bay. While little damage to brick veneer was reported, one particular property which was under construction exhibited significant damage in the veneer cladding (Figure 2). The veneer was 95 mm thick split concrete brick in this instance. It appeared that the bond between the bricks had failed sufficiently that individual bricks were ready to fall from the veneer.

Figure 2. View of the damaged veneer on a Gisborne house

1.4 Literature survey

The review of previous investigations both in New Zealand and overseas is presented in BRANZ *Study Report 190* (Thurston and Beattie 2008b).

1.5 Outline of test report

This report describes the construction and instrumentation of a full-scale concrete brick veneer specimen incorporating both face-loaded and in-plane loaded veneer elements, complete with openings. It goes on to describe the behaviour of the specimen over the range of dynamic shake tests imposed on it. The plan dimensions of the specimen were identical to those of the dynamically tested clay brick veneer specimen (Thurston and Beattie 2008b).

1.6 The bricks, brick ties and mortar used in the testing

Details of the brick construction used in the test are given below. The choice of these products is expected to have a significant influence on the test results. Further tests are required before general applicability can be determined.

The concrete bricks used had dimensions 290 mm long x 160 high x 70 mm wide and were donated by Firth Industries. A portion of veneer was weighed during room dismantling as shown in Figure 3. It was concluded that the assembled veneer weighed 125 kg per square metre of face area.

Figure 3. Measuring weight of veneer

Unlike the clay used in the Phase 2 investigation, the concrete bricks used did not have vertical holes. Thus, no mortar “dowels” formed between concrete bricks and because of this they were more susceptible to sliding on the mortar joints.

The bricks were laid by tradespeople using Dricon Trade Mortar, with approximately 10 mm thick mortar being used between the bricks on both horizontal and vertical surfaces. This mortar was stated as complying with NZS 4210 (SNZ 2001) for masonry construction.

Commonly available hot-dipped galvanised, 85 mm long, 70 series, Eagle brand brick ties were fully encapsulated within the mortar. The ties were stated to be rated “heavy duty” to the AS/NZS 2699.1 standard.

The ties were fixed to the face of the timber studs using galvanised, self-drilling, 35 mm long Tek screws which are supplied with the ties.

2. CONSTRUCTION OF TEST ROOM

The construction of the test room was essentially identical to that used in Phase 2 of this investigation for the clay brick veneer room (Thurston and Beattie 2008b) to enable a valid comparison of performance to be made. The mass added to the top of the room was also identical to that used in Phase 2.

The test room was a single-storey nominally 2.4 m high room of dimensions 4.07 m long (in the direction of shaking) by 2.51 m wide. It had plasterboard-lined LTF walls, brick veneer on all four sides and a timber-framed plasterboard-lined ceiling. Figure 4 shows a plan cross-section of the test room, defines the labelling of the veneer panels and names the sides and ends that will be referred to later in the report. The brick veneer is shown shaded.

Figure 5 and Figure 6 are elevations of the LTF walls showing framing and the added weights to simulate the inertial mass of the roof.

Figure 7 and Figure 8 are photographs showing the veneer on the two side walls and some of the instrumentation. Figure 9 shows a cross-section through the base of the walls. The veneer was separated from the LTF by 50 mm. It is a requirement of NZS 3604 (SNZ 1999) that the cavity is between 40 and 70 mm wide. 50 mm is commonly used by brick-layers to keep within the tolerances.

As shown in Figure 4, Panels A, B, E and G were the panels on the long side of L-shaped corner veneer elements, whereas Panels J, C, D and H were respectively the adjacent panels on the end walls. However, the ends of Panels A and B (remote from the corner) finish adjacent to a window, whereas the ends of Panels G and E (remote from the corner) finish at free ends, which is the situation at doorway openings. Panel F is referred to as an isolated pier and is free to either slide or rock without encumbrance from the adjacent panels. The size of the test room did not allow for piers between windows to be incorporated.

The wall lining was 10 mm Elephant Multiboard plasterboard. As there was doubt about the ability of the shake table system to induce failure of the test room, the lining was only lightly attached to the framing as shown in Figure 10(a). The reasons for the chosen lining strength are discussed in Thurston and Beattie (2008b). The lining was placed vertically, and the joints between sheets (including at room corners) were not reinforced with tape or stopped to ensure there was no additional strength due to "systems effects". However, all ceiling joints (including the junction between walls and ceiling) were reinforced with paper tape and stopped to ensure representative load transfer from the ceiling to the walls.

Other details of the construction are set out below:

2.1 General

- The outside plan dimensions of the brick veneer were 4.07 m x 2.51 m.
- Side 1 incorporated a window opening. Each of the end walls had a simulated door opening and there were two simulated door openings on Side 2. No doors or windows were installed in the openings.
- As the shake table size was limited, the door openings were made very narrow. Larger width door openings, while keeping other dimensions the same, are not expected to have significantly changed the total room lateral resistance.
- The LTF walls were constructed from 90 x 45 mm kiln-dried radiata pine with maximum stud spacing being 600 mm.

2.2 Brickwork

The veneer consisted of 13 courses of concrete bricks placed in running bond. Galvanised steel ties were placed on top of the 1st, 3rd, 5th, 7th, 9th, 11th and 12th courses of bricks and were screwed to timber studs which were at a maximum of 600 mm horizontal spacing. The bricks were interlocked at the corners as shown in Figure 11.

Details of the concrete bricks, brick ties and mortar are presented in Section 1.6.

The column of brickwork between the veneer window openings is referred to as a "pier" in this report. Isolated brick veneer walls which are completely separated from other brickwork are referred to as "isolated veneer panels". The brickwork on both sides of a corner tends to act as a single unit and is referred to as an "L-shaped corner veneer element".

2.3 Foundations

The bottom plates of the LTF walls were bolted to an inverted 75 mm deep steel channel strong floor at each end of each wall panel, as shown in Figure 9. The steel channel was rigidly attached to the top of the shake table.

The concrete brick veneer was constructed on a steel channel filled with concrete to provide a bed for the veneer, as shown in Figure 9.

3. INSTRUMENTATION

Potentiometers were used to continuously measure the following movements during the applied shake table motion:

- The uplift deflection of each veneer panel relative to the shake table at each end of each side panel (i.e. Panels A, B, E, F and G) as shown in Figure 7. Note that care was taken to ensure these measurements included all crack openings.
- The out-of-plane horizontal deflection of the brickwork relative to the LTF on either side of the opening at each end wall.
- The in-plane horizontal deflection of the top of the brick veneer relative to the LTF of each side panel as shown in Figure 7.
- The in-plane horizontal deflection of the second brick from the bottom of the brick veneer relative to the shake table at mid-length of each side panel.
- The horizontal deflection of the ceiling relative to the shake table at the mid-width of the room. This was achieved by constructing a stiff/rigid 1.8 m long wall with a height just short of the ceiling and fixing this to the table, and then by measuring the deflection between the top of this wall and the ceiling.

A ruler was used to measure residual uplift and horizontal slip of the LTF walls at many places after each shaking test phase (no movement was detected).

The applied actuator load and deflection were also continuously measured.

A video camera on each side visually recorded behaviour during the testing.

Accelerometers were used to measure the horizontal acceleration of the table, ceiling, and the top of the two side brick veneer walls.

4. SELECTION OF INERTIAL MASS, SPECIMEN STRENGTH AND BRACING DEMANDS

Full details of the selection of inertial mass, specimen strength and bracing demands are contained in Thurston and Beattie (2008b), as the two specimens were identical except for the different veneer materials. However, a summary of these are presented below.

4.1 Room weights

The estimated contributing house roof and ceiling weight, W_h = 34.93 kN.

The calculated ceiling weight of the room on the shake table, W_c = 2.08 kN.

Therefore, the required added weight = $W_h - W_c$ = 32.85 kN.

4.2 Shake table limitations

The limitations on the dynamic response of the shake table meant that the following maximum input acceleration levels could be expected:

0.16g at 1 Hz, 0.32g at 2 Hz, 0.48g at 3 Hz, 0.64g at 4 Hz.

4.3 Predicted LTF wall racking strength

The predicted strength of the LTF wall was 10.0 kN with the linings fixed at 600 mm centres to the framing.

4.4 Bracing wall demand loads and ratios with wall strengths

The calculated NZS 3604 bracing demand for Zone A for the test room was 14.05 kN. The ratio of the NZS 3604 demand load to the design strength is therefore = 14.05/10 = 1.41.

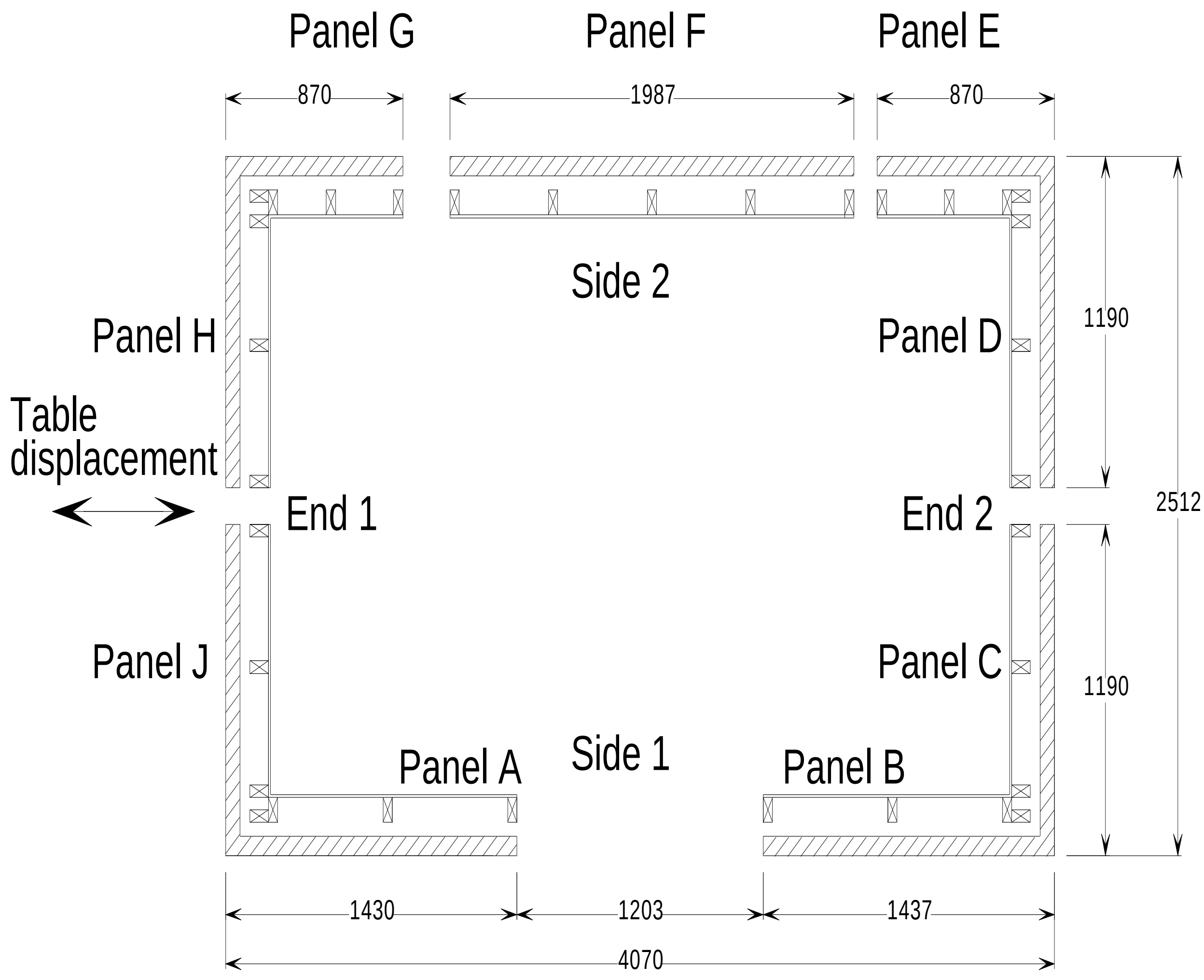
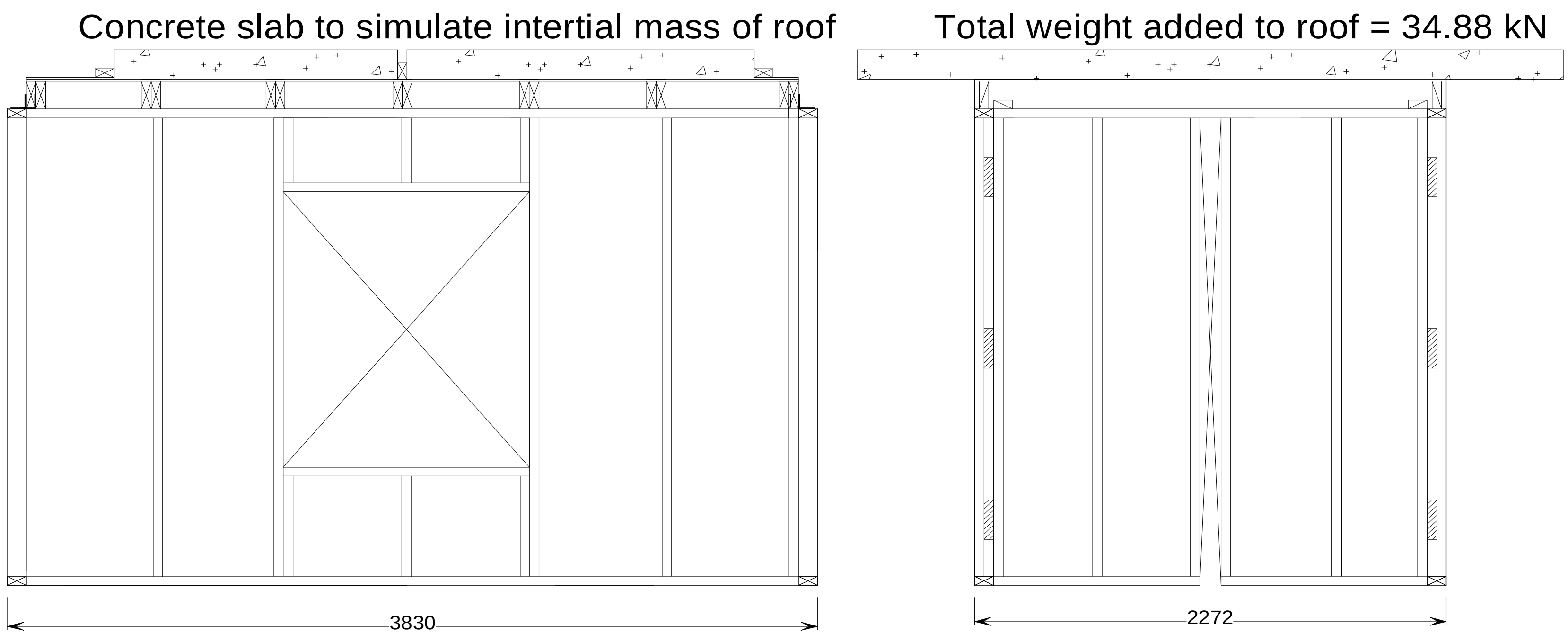


Figure 4. Plan section showing dimensions and labelling system



Side1 wall framing

Wall framing (both ends)

Figure 5. LTF framing on Side 1 and the end walls and added roof weights

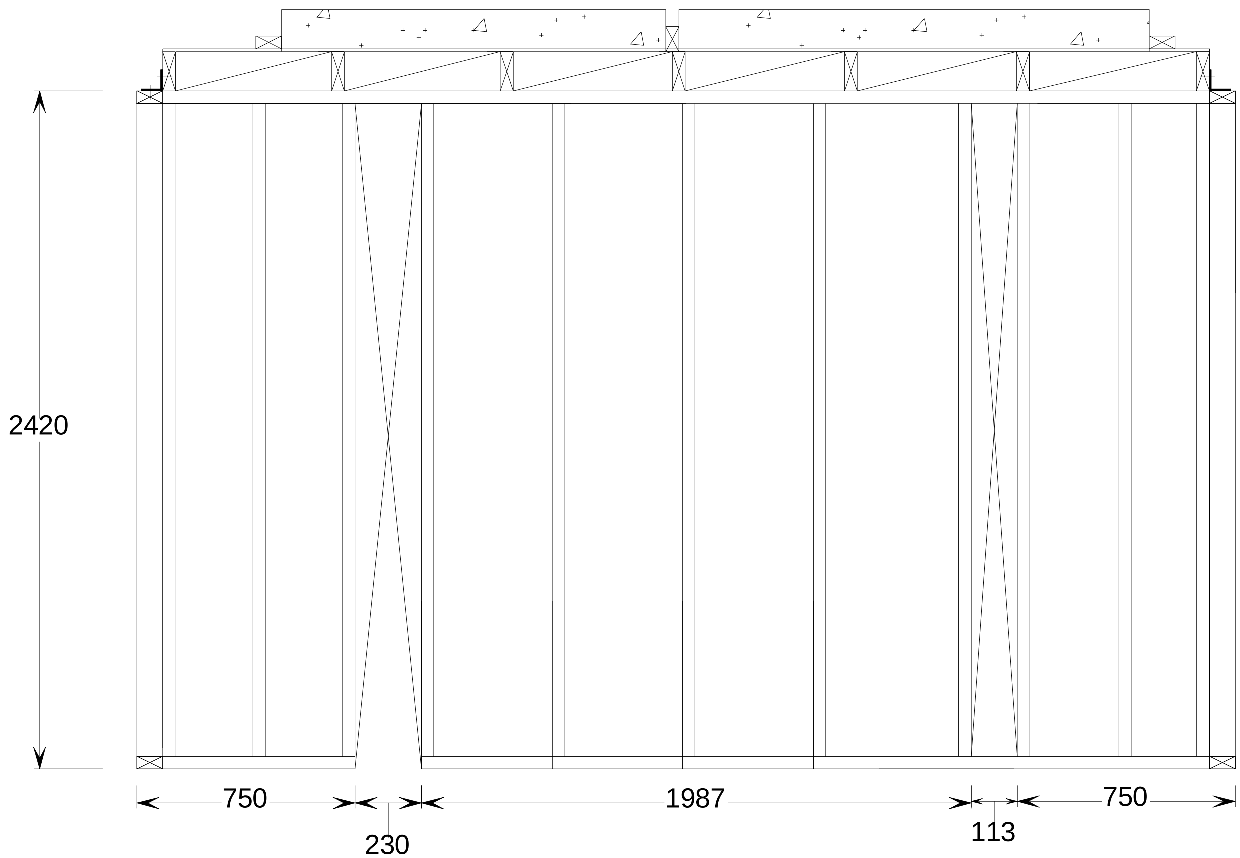


Figure 6. LTF framing on Side 2

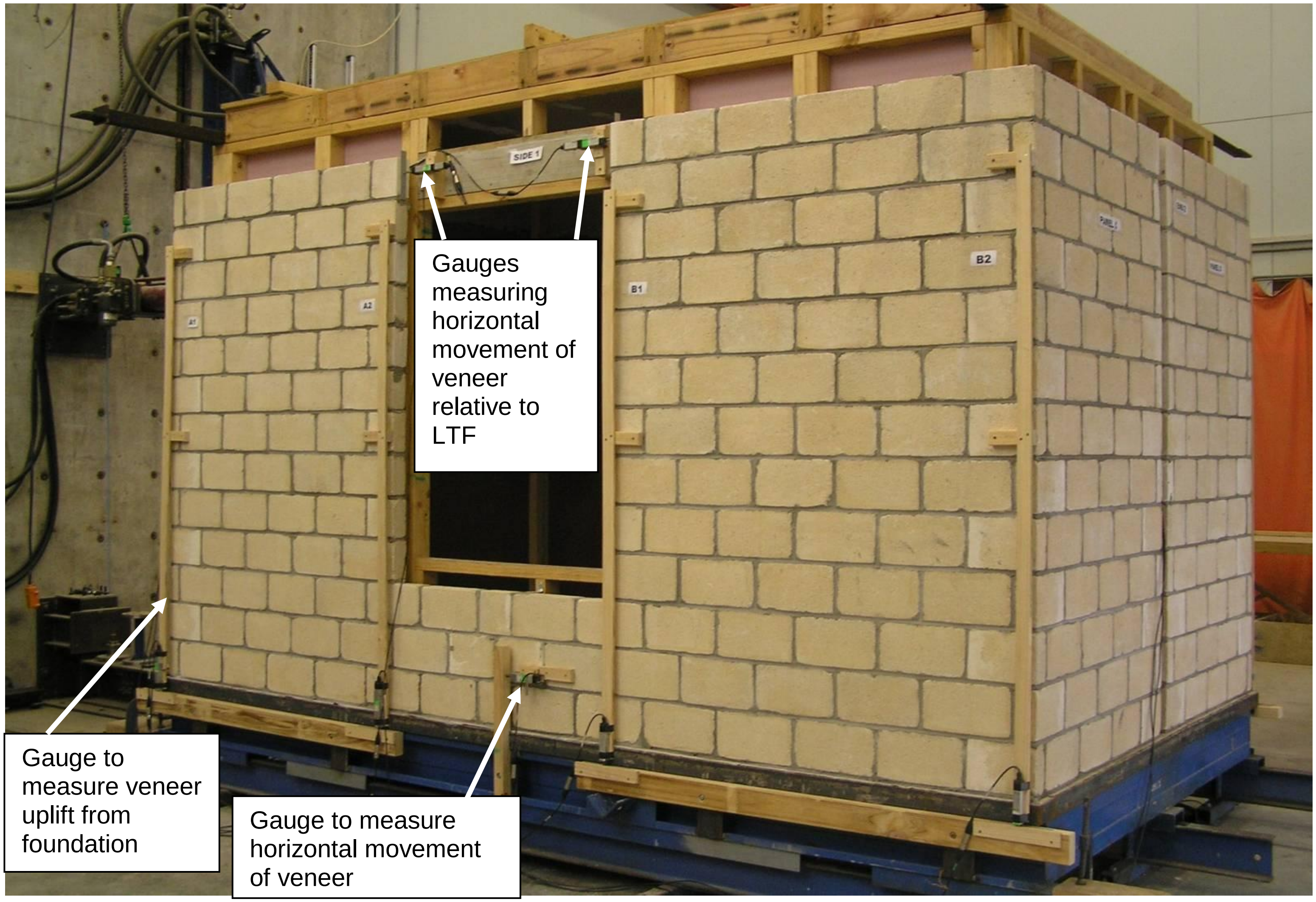


Figure 7. View of Side 1 before testing

Figure 8. View of Side 2 before mass added and testing

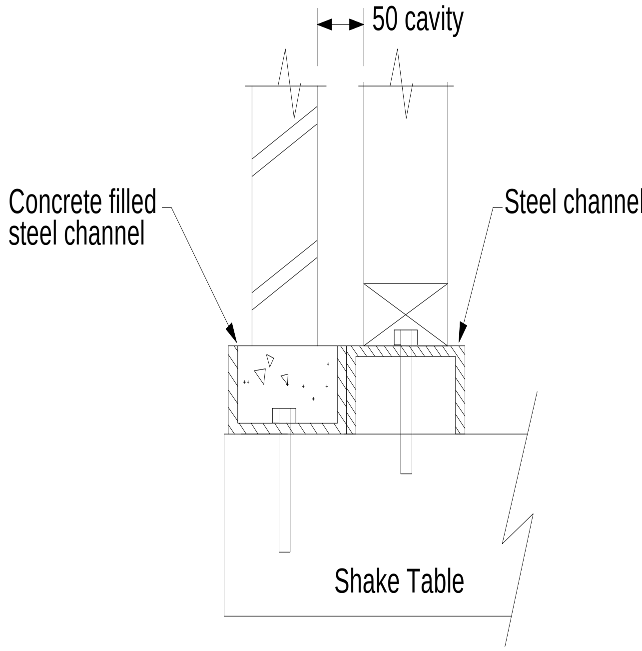
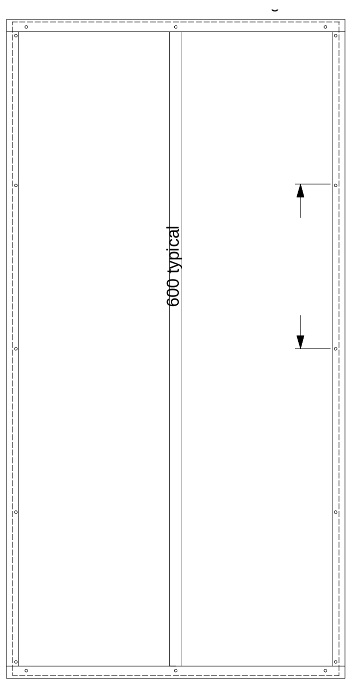
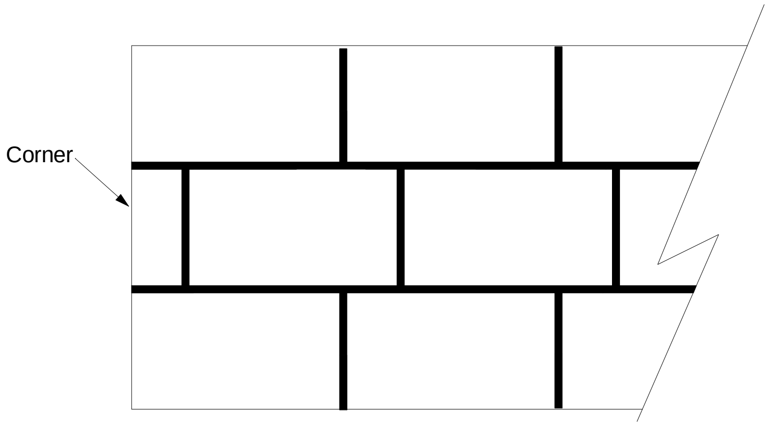


Figure 9. Cross-section through bottom of veneer and LTF walls

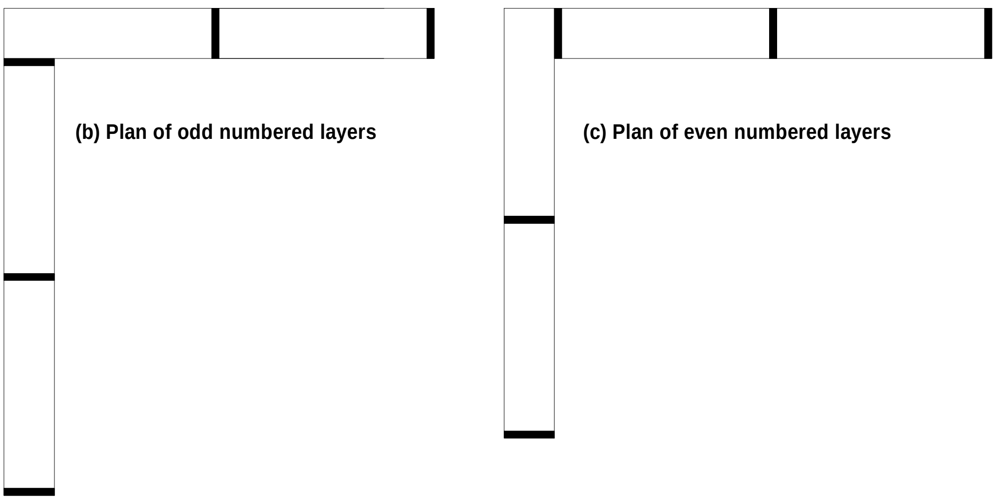


All screws were 32 mm x 6g drywall plasterboard screws.
There were no fixings in the body of the sheets

Figure 10. Typical fixing of plasterboard lining



(a) Elevation of veneer layout at a corner



(b) Plan of odd numbered layers

(c) Plan of even numbered layers

Figure 11. Interlocking of brick veneer at corners

5. HISTORY OF IMPOSED SHAKING AND ASSOCIATED OBSERVATIONS

5.1 Explanation of Table 1 – tests performed

The history of shaking applied to the shaking table is summarised in Table 1. This gives the frequency and displacement of the imposed simple harmonic motion as well as the derived velocity and acceleration, from simple harmonic motion formulae. The peak applied simple harmonic acceleration was 0.23g.

Table 1 also summarises the measured amplitude of ceiling horizontal deflection relative to the shake table. For instance in Test R16, 10 cycles of sinusoidal motion were imposed on the table at 2 Hz. The table displacement was ±14.23 mm and the peak ceiling deflections relative to the table were measured as ±46.0 mm. Note 3 at the bottom of the table comments on this test.

Table 1. Summary of shaking imposed and ceiling response

Test name	Freq. (Hz) *	Table Movement ±(mm)	Calculated Velocity (mm/sec)	Calculated Accel. (g)	Number of cycles	Ceiling defl. relative to table ±(mm)	Notes
R1	0.5	59.95	188.3	0.06	10	0.30	
R2	1	35.49	223.0	0.14	10	1.00	
R3	2	8.51	106.9	0.14	10	2.33	
R4	3	3.61	68.1	0.13	10	1.77	
R5	4	1.34	33.7	0.09	10	2.35	
R6	5	1.07	33.7	0.11	20	4.84	
R7	6	0.74	27.8	0.11	20	2.62	
R8	7	0.58	25.6	0.11	20	1.74	
R9	8	0.53	26.5	0.14	20	1.40	
R10	7 to 6.3	0.53			≈ 70	1.40	
R11	6.3 to 5	0.84			≈ 90	2.66	
R12	5 to 4	1.09			≈ 55	5.26	
R13	5	1.48	46.5	0.15	20	4.28	
R14	4	2.15	54.0	0.14	20	9.1	1
R15	3	5.75	108.4	0.21	10	20.5	2
R16	2	14.23	178.8	0.23	10	46.0	3
R17	1.5	19.50	183.8	0.18	10	58.4	4
R18	1	29.96	188.3	0.12	10	85.6	5

* Sinusoidal shaking frequency. If a sweep, then the range of frequencies is shown.

Notes:

1	Minor cracking and panel rocking detected but cracks were hard to see when shaking stopped. Significant damage had occurred in the plasterboard linings at fastener attachment points.
2	Cracking due to panel rocking now easily seen during shaking but again cracks were hard to see when shaking stopped. Severe lining damage.
3	Linings had mainly fallen off or were only loosely attached to the LTF walls. During shaking the cracks had opened along mortar courses, but the veneer did not slide along mortar courses and there was little observable damage when the shaking had stopped.
4	Most panels had one or two horizontal mortar courses on which the bricks slipped, thereby partially dislodging corner bricks. After shaking had stopped the perpend joint between some bricks had opened up to 10 mm (in particular the corner bricks).
5	Perpends between some blocks opened to 50 mm. Six bricks fell at corners. Many bricks, especially near corners, had completely separated from adjacent bricks. Many rows of bricks were slipping on whole mortar courses.

In addition, the 10 cycles of Test R18 were repeated three times and these are called R19, R20 and R21 for reference in the photographs.

5.2 Observations

Crack patterns are drawn in Figure 13 to Figure 14. General photographs of the veneer cracking, taken when the shaking had stopped, are shown in Figure 15 to Figure 21.

No veneer cracks were observed in Tests R1 to R11 (where the motion of the LTF ceiling relative to the shake table was less than ±5 mm). However, some reduction in room stiffness/natural frequency did occur as discussed in Section 7.1. Very fine mortar cracks were noted after Test R12 (as shown in Figure 13 and Figure 14) where the ceiling motion relative to the table was ±5.3 mm.

In Test R14, which was run at 4 Hz, the recorded LTF ceiling deflections were ±9.1 mm and rocking in all four “L-shaped veneer corner elements” was clearly visible. The relative deflection of the ceiling with respect to the shake table increased to ±20.5 mm in Test R15 at 3 Hz. The damage to the lining fixings was noted to be severe after this test. In Test R16, at 2 Hz, the recorded LTF ceiling deflections were ±46 mm. This movement was almost entirely accommodated by rocking of the veneer panels above the base cracks.

At this stage, although fine cracks were observed as shown in Figure 13 and Figure 14, the veneer panels appeared to act as single units, there was no slippage along mortar joints and cracks closed at the end of these tests. The only visible damage after the tests was fine mortar cracks which could be repaired by re-pointing.

In Test R17 significant slipping occurred along some horizontal mortar joints, and at some levels at the room corners the bricks were dislodged sideways leaving up to a 10 mm gap in the perpends between bricks. Several major cracks greater than 10 mm wide occurred in each veneer panel during shaking. However, at the end of testing an engineer inspecting damage after an earthquake may have considered that the veneer could simply be repaired by re-pointing as there was still little evidence of the distress that the veneer had experienced.

In Test R18 slippage occurred along several horizontal mortar courses within all L-shaped corner veneer elements. Many bricks near corners completely separated from adjacent bricks, twisting out-of-plane, and were only tenuously held in position as shown in Figure 15 and Figure 16. In other places, gaps between adjacent bricks were up to 50 mm. The bricks near the bottom of Panel G separated into individual units with two bricks falling from this panel. Part-way through the test, Panel F stopped rocking and the motion was absorbed almost entirely by horizontal slippage on the mortar course above the 7th brick. This horizontal slippage was approximately ±70 mm. A brick fell at the opening immediately below this level.

At this stage each panel had a multitude of cracks and it was considered that the entire veneer would have needed to be demolished if it had been in this condition in a real building.

Tests R19, R20 and R21 were simply repeats of Test R18 for demonstration purposes. As there was no data recording, these tests are not included in Table 1. The veneer deteriorated significantly in these 30 seconds of testing, as discussed in Section 6.

At the end of the testing, during dismantling, it was noted that two ties near the top window opening of Pier A had broken and several adjacent ties had fatigue cracked almost right through. At the top of Panel F several adjacent ties had fatigue cracks almost right through. Elsewhere the ties were uncracked. All were still firmly screwed to the timber framing and embedded within the brick mortar.

Figure 12 relates the cumulative number of cycles of ceiling motion greater than discrete ceiling deflections given by the x-axis. For instance, there were 40 cycles where the ceiling deflection was ±20 mm or more. In addition to the testing summarised in Table 1, the 10 cycles of Test R18 was repeated three times and these are called R19, R20 and R21 for reference in the photographs given in this report.

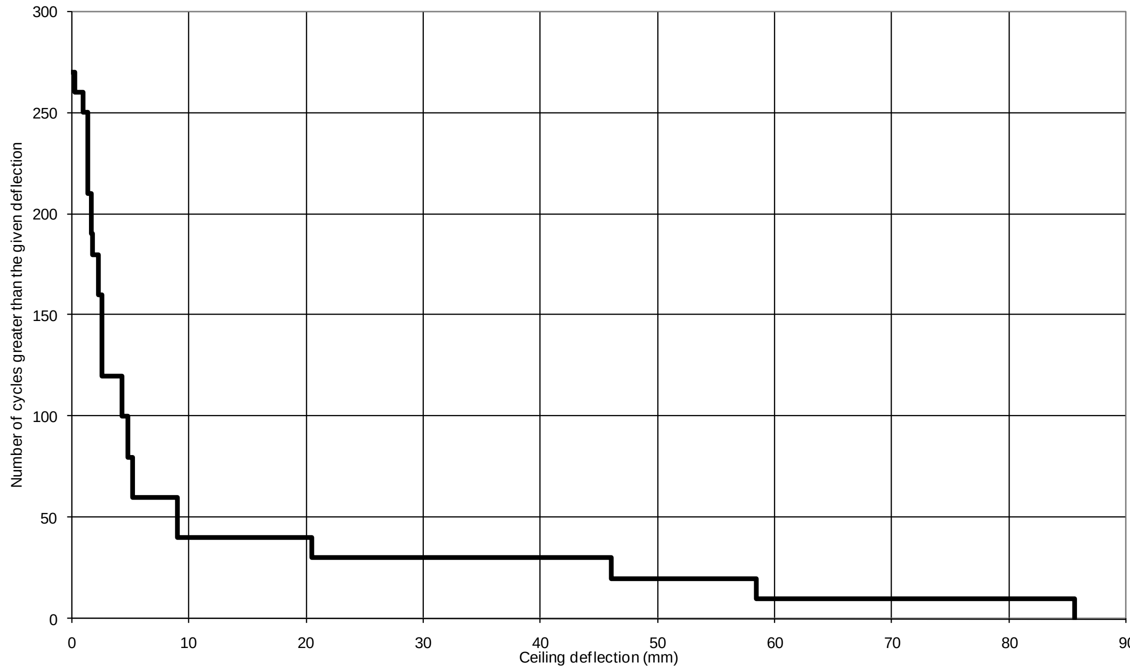


Figure 12. Number of excursions beyond discrete deflections

6. VENEER CRACKING

The observed veneer cracking at the end of Tests R12, R16 and R18 is shown in Figure 13 for Side 1 and Figure 14 for Side 2. Photographs are given in Figure 13 to Figure 21. A few concrete bricks completely dislodged and fell to the floor. These are shown hashed.

All four L-shaped corner veneer elements and the single isolated pier (Panel F) rocked about alternate ends of the cracks shown during the applied backward and forward motion. Unlike the shaking tests on the clay brick veneer (Thurston and Beattie 2008b), increased rocking resulted in panels sliding on horizontal mortar courses and a multitude of additional mortar cracks formed. In places, the veneer fragmented into separated bricks. The brick interlocking at corners was not as effective as in the clay brick construction and brick sliding and separation occurred mostly at panel corners or free ends. However, there was little evidence of the room end wall brick veneer having significant movement in the out-of-plane direction relative to the LTF.

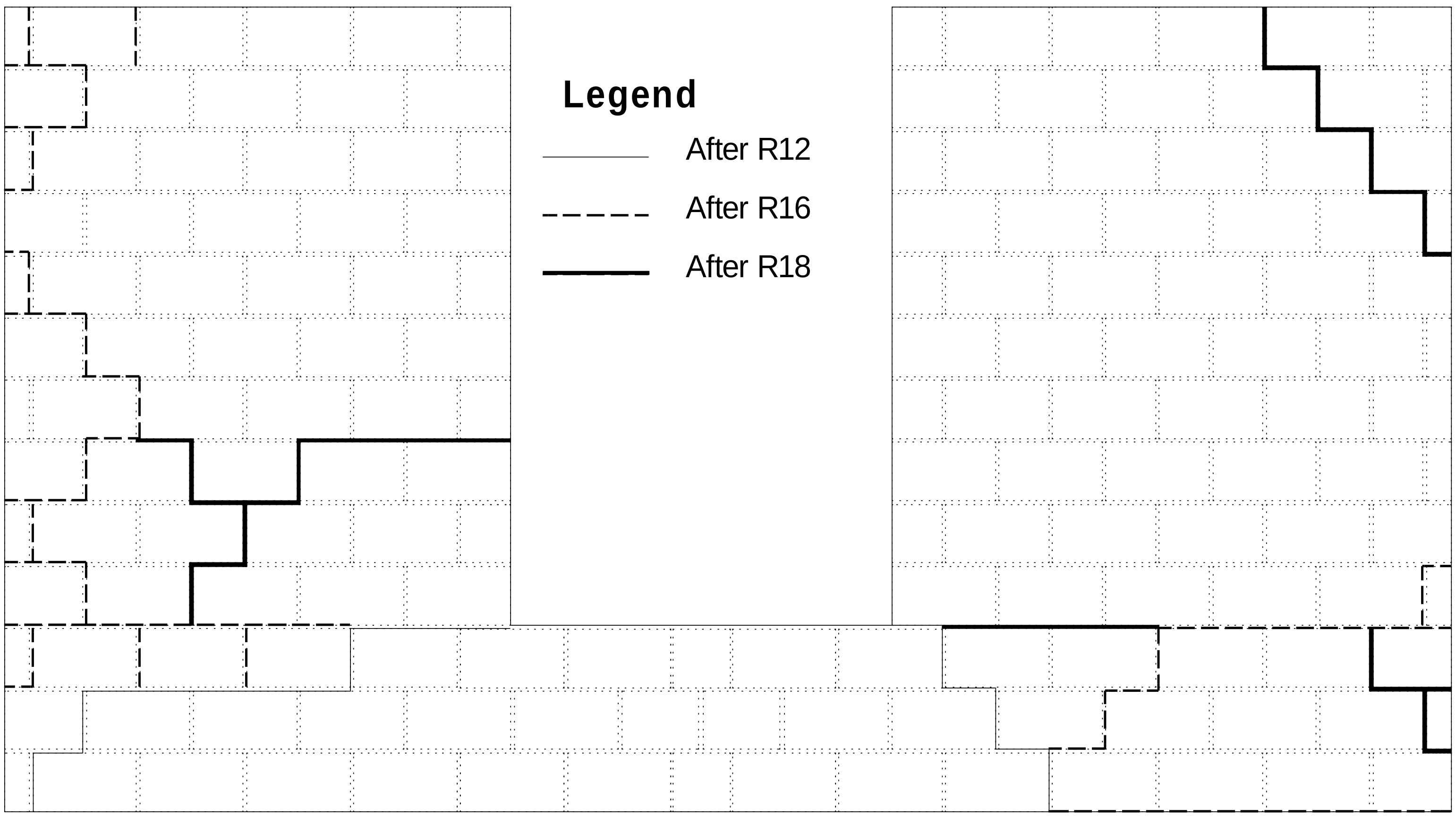


Figure 13. Cracking in Side 1

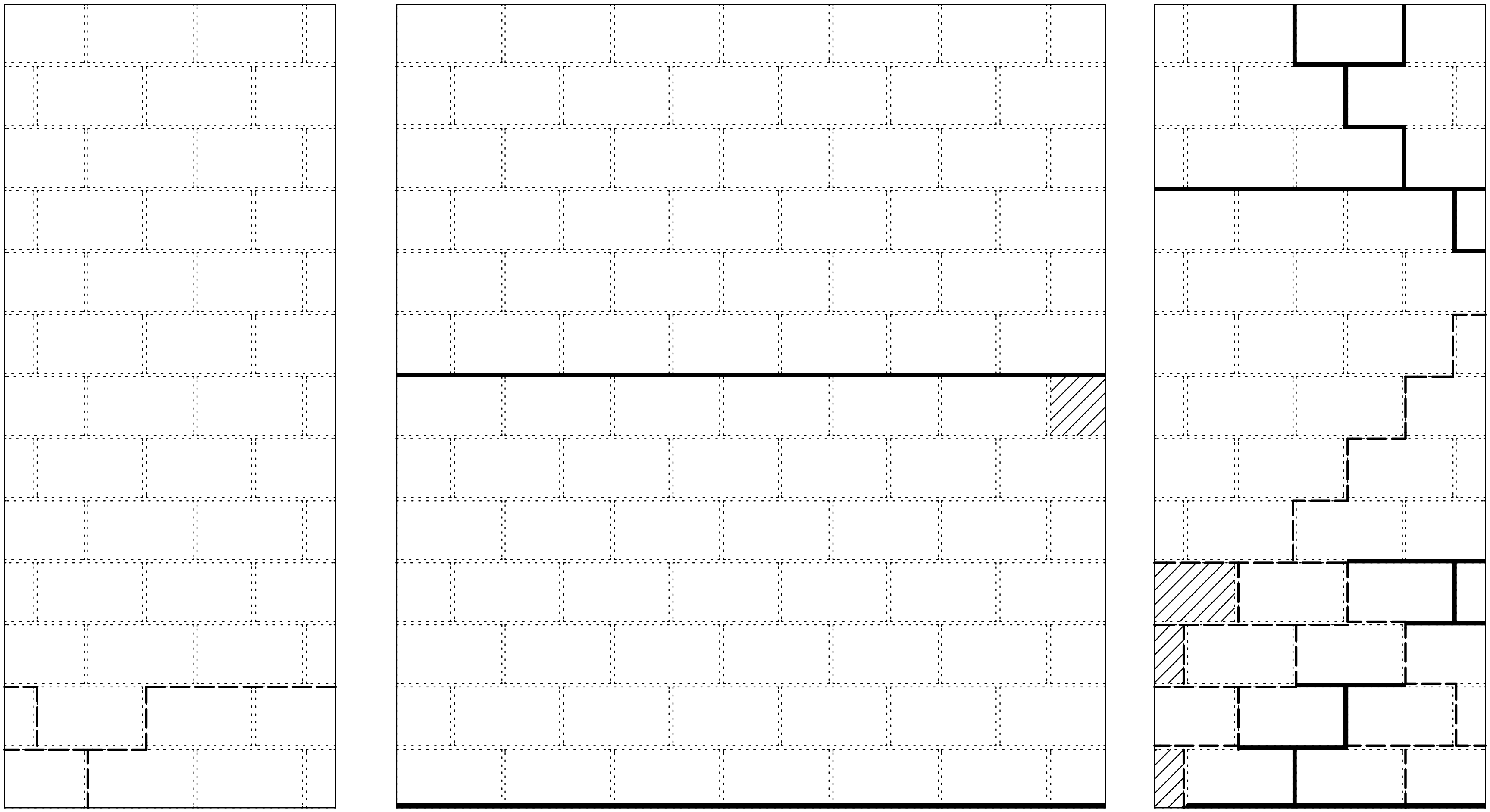


Figure 14. Cracking in Side 2 (legend as given in Figure 13)

(a) Bottom corner of Panel A

(b) Top corner of Panel A

Figure 15. Panel A after Test R19

(a) Bottom corner of Panel B

(b) Top corner of Panel B

Figure 16. Panel B after Test R19

Figure 17. Side 2 after Test R19

(a) Bottom corner of Panel E

(b) Bottom corner of Panel G

Figure 18. Side 2 after Test R19

Figure 19. Side 1 after Test R21

Figure 20. Corner of Panel B after Test R21

Figure 21. Side 2 after Test R21

7. MEASURED MOVEMENTS IN SHAKE TABLE TESTS

7.1 Amplification of motion and room natural frequency

If an elastic system with a single natural frequency is subjected to simple harmonic motion at a range of frequencies it will show high amplification of movement at the natural frequency. This is a method of identifying the natural frequency.

Figure 22 plots the amplification ratio (= amplitude of ceiling displacement relative to the table divided by the amplitude of table displacement) versus applied harmonic frequency for all tests. A line joining circles gives the results of the first nine tests (R1 to R9) which indicate that the natural frequency of the lined room was approximately 5 Hz. Then three sweep runs were made (R10 to R12). The first of these is ignored as it did not give a peak response. In R11 an apparent peak occurred at 5.1 Hz with lower amplification than that previously found in the first nine tests (R1 to R9) indicating the room stiffness (and hence natural frequency) had reduced. The peak in R12 was at 4.1 Hz which is considered to be the natural frequency of the room in this reduced stiffness state.

The subsequent tests were broken into two sets of five cycles. The results for the first set are shown in Figure 22 with a "+" symbol and the results for the subsequent five cycles are shown with a "Δ". In Tests R14 to R18 the amplification ratio is greater than for tests R1 to R5 at corresponding frequencies, as the room has lost stiffness, the natural frequency decreased, and the frequencies of Tests R14 to R18 were closer to room resonance.

The repeat tests show a decrease in the amplification ratio due to further room stiffness reduction as the lining fixing deteriorates further. The room natural frequency for the case with ineffective linings can best be found by looking at these test results. In the repeat test of R16, and in both Tests R17 and R18, the lining had become ineffective. This data is encased in an oval in Figure 22. The "Δ" of R16 may be a slightly higher amplitude than it would be for the no-lining case. In total, these three tests indicate a natural frequency less than 1.5 Hz and probably close to 1 Hz.

In summary, the natural frequency of the room when the linings became ineffective was taken as approximately 1 Hz. The room response was not particularly sensitive to the exact value. The natural frequency when the linings were fully effective was taken as 5 Hz.

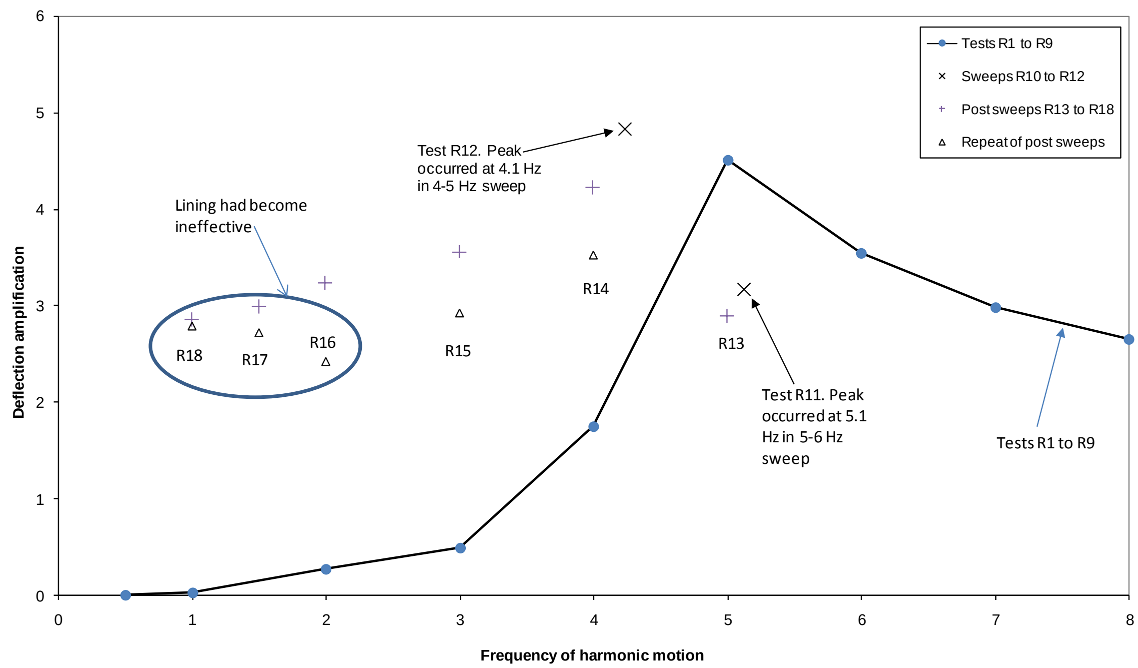


Figure 22. Deflection amplification ratios used to determine room natural frequency

7.2 Rocking deflections

The horizontal in-plane differential movement between the top of the brick veneer and the adjacent LTF frame was measured for each side wall panel. This measurement is referred to as tie movement as it is effectively the distortion of the tie because no slip between tie and mortar or between tie and LTF frame was observed. The cyclic amplitude of tie movement (i.e. X where the movement is ±X mm) is plotted in Figure 23. It can be seen that the tie movement was small for each of the L-shaped corner veneer elements except for Test R18 (where the maximum ceiling displacement was ±86 mm). It is considered that the Test R18 result should be discarded as the observed horizontal slip of the bricks probably influenced the results. The small tie movement for other tests corresponded to the observations and subsequent theory by Thurston and Beattie (2008a), where it was considered that the ties connecting the end walls to the LTF effectively forced the L-shaped corner veneer elements to deform the same distance as the LTF wall and thus the in-plane tie deformation would be negligible.

Large tie distortion of approximately 50% of the ceiling movement occurred in the isolated veneer Panel F, even for Test R18, where a large brick slip occurred along the mortar course above the 7^{th} brick-layer up the panel.

The horizontal displacement, Δ, of the top of a brick veneer panel induced by rocking is given by:

$$\Delta = (V_A - V_B) \times H / L_{AB} \text{ ....}$$

Where H is the panel height above the pivot point
V_A, V_B are the vertical movements measured at the two panel ends, A and B
L_{AB} is the distance between points A and B

The amplitude of panel rocking deflection is plotted in Figure 24 versus the measured ceiling displacement. The rocking deflection of the L-shaped corner veneer elements (Panels B, E and A) was approximately 0.8 times the ceiling deflection relative to the table. The exception is Panel B in Tests R17 and R18 where it is believed that the cracking at the top of the panel distorted the readings. Panel F, the isolated pier, had significantly less deflection as predicted by the theory, which postulates that horizontal tie distortion will be smaller for L-shaped corner veneer elements than for side panels and thus these panels will deflect less than corner panels.

7.3 Base slip

Figure 25 plots the amplitude of panel horizontal base slip versus the applied table displacement for all panels. The base slip of all panels was low, including the isolated veneer Panel F. As this was measured from the 2^{nd} to bottom brick to the base, some of the measured movement would have been due to panel rocking. Panel F exhibited the most slip but this reduced in Test R18 (LTF deflection of ±86 mm) where Panel F sliding at Level 7 absorbed much of the ceiling movement.

7.4 End wall veneer out-of-plane movement relative to the LTF

The horizontal out-of-plane differential movement between the top of the brick veneer and the adjacent LTF frame was measured for each end wall panel. This measurement is referred to as veneer out-of-plane movement and is effectively the compression buckling of the tie because no slip between tie and mortar or between tie and LTF frame was observed and no tie extension was observed. The peak veneer out-of-plane movement from each test is plotted in Figure 26. The negative values mean the veneer moves closer to the LTF. It can be seen that the movement is generally small and similar in magnitude to the in-plane movement of the L-shaped corner veneer elements plotted in Figure 23 (as would be expected from the theory by Thurston and Beattie (2008a)).

The range of movement in each test is plotted in Figure 26. For example, in Test R17 the top of Panel D End 2 was closer to the LTF by between 0.31 and 3.01 mm during the test. Thus, a point at -3.01 mm is plotted in Figure 26 at the ceiling displacement of 58.43 mm and a point at 2.71 mm (=3.01-0.31) is plotted in Figure 27 at the ceiling displacement of 58.43 mm. The measured gap between the walls increased only slightly during any of the tests. A comparison of the plots in Figure 27 and Figure 27 shows that the ties never fully straightened after buckling.

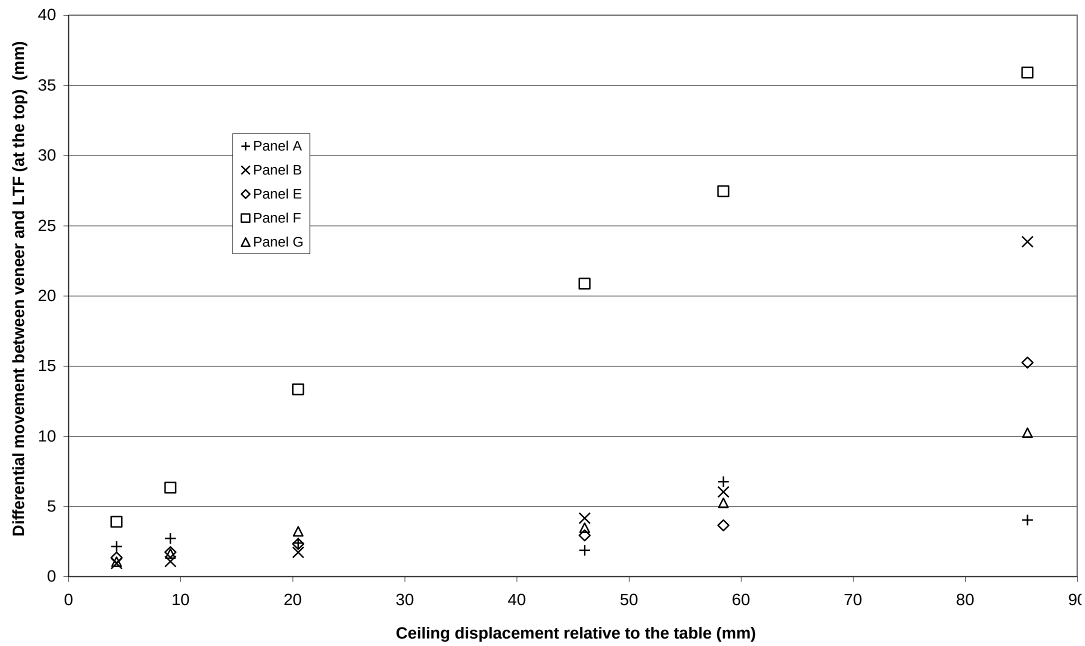


Figure 23. Differential displacement between top of veneer and LTF versus ceiling displacement (= tie distortion)

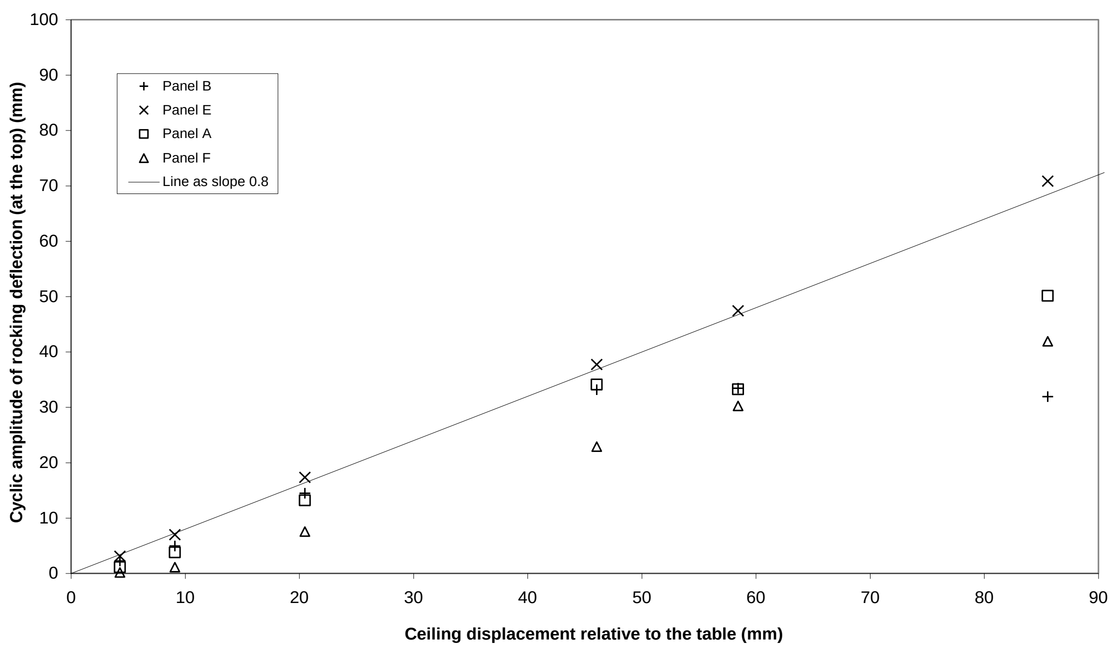


Figure 24. Panel rocking deflection versus ceiling displacement

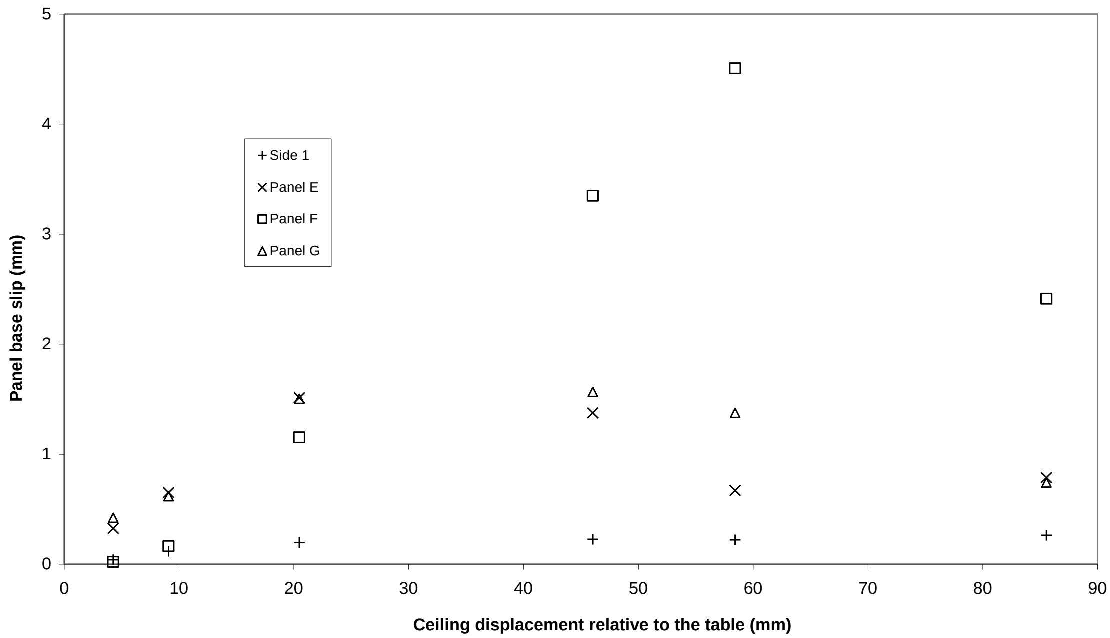


Figure 25. Panel base slip versus ceiling deflection

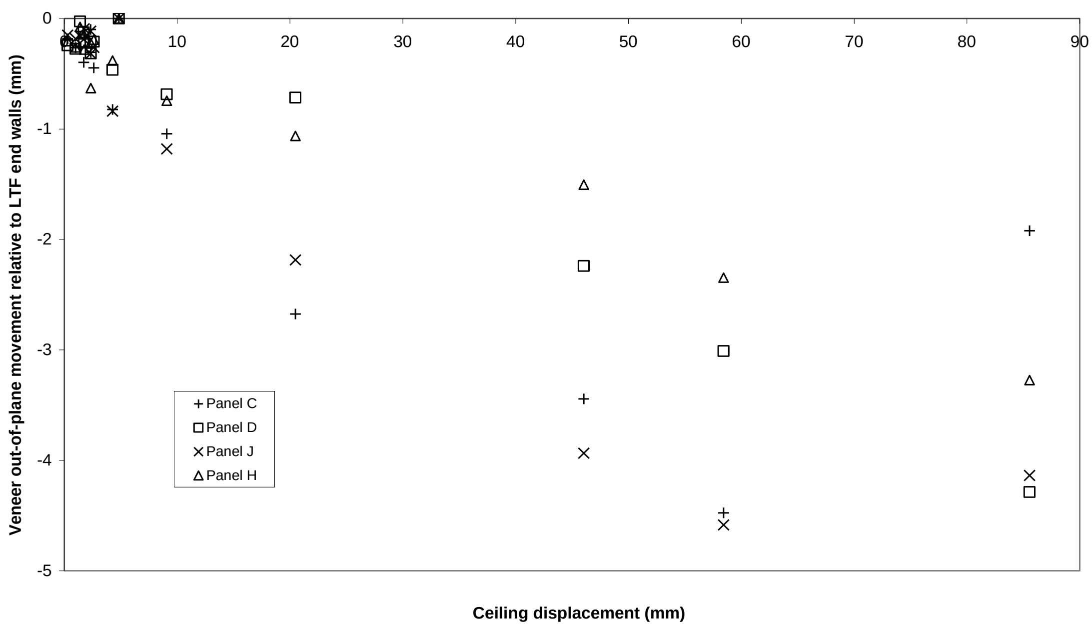


Figure 26. Maximum veneer out-of-plane movement from initial zero

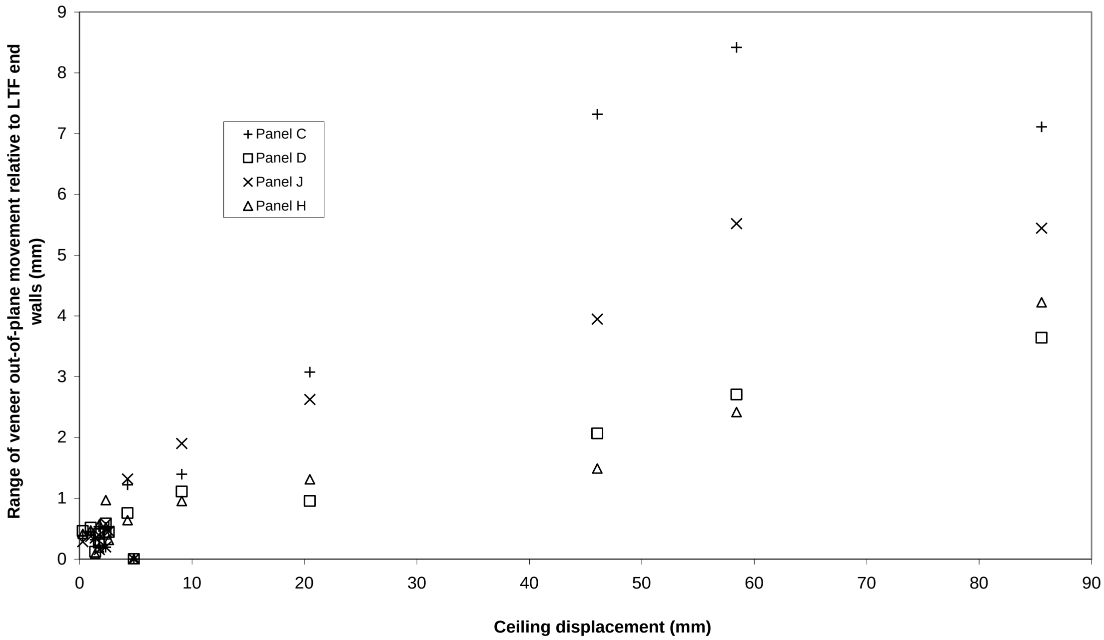


Figure 27. Range of out-of-plane veneer movement in each test

8. MEASURED ACCELERATIONS IN SHAKE TABLE TESTS

Typical plots of measured table, ceiling and top of veneer acceleration time histories are shown in Figure 28 to Figure 30 respectively. It can be seen that the table accelerations are far from sinusoidal and the ceiling accelerations are smoother and closer to sinusoidal. The impact as the rocking veneer hits the table has apparently affected the accelerometers.

The dynamic amplification factor is defined as the ratio of the acceleration of the excited mass to the acceleration of the table (or ground) and is an important parameter as it relates the seismic inertia forces to the accelerations experienced. As the records of accelerations exhibited spikes, particularly when the rocking veneer panels impacted the table, the dynamic amplification factor is taken as the ratio of the RMS (root mean square) of the measured accelerations.

Figure 31 plots the ratio of ceiling RMS acceleration to table RMS acceleration versus ceiling deflection. It can be seen that there is a considerable scatter of results. For ceiling deflections greater than 10 mm, a best fit straight line gave a ratio of 1.72 for 10 mm ceiling deflection and a ratio of 1.50 for 80 mm ceiling deflection.

Figure 32 plots the ratio of top of veneer RMS acceleration to table RMS acceleration versus ceiling deflection. Again it can be seen that there is a considerable scatter of results. For ceiling deflections greater than 10 mm, a best fit straight line gave a ratio of 2.75 for 10 mm ceiling deflection and a ratio of 2.35 for 80 mm ceiling deflection.

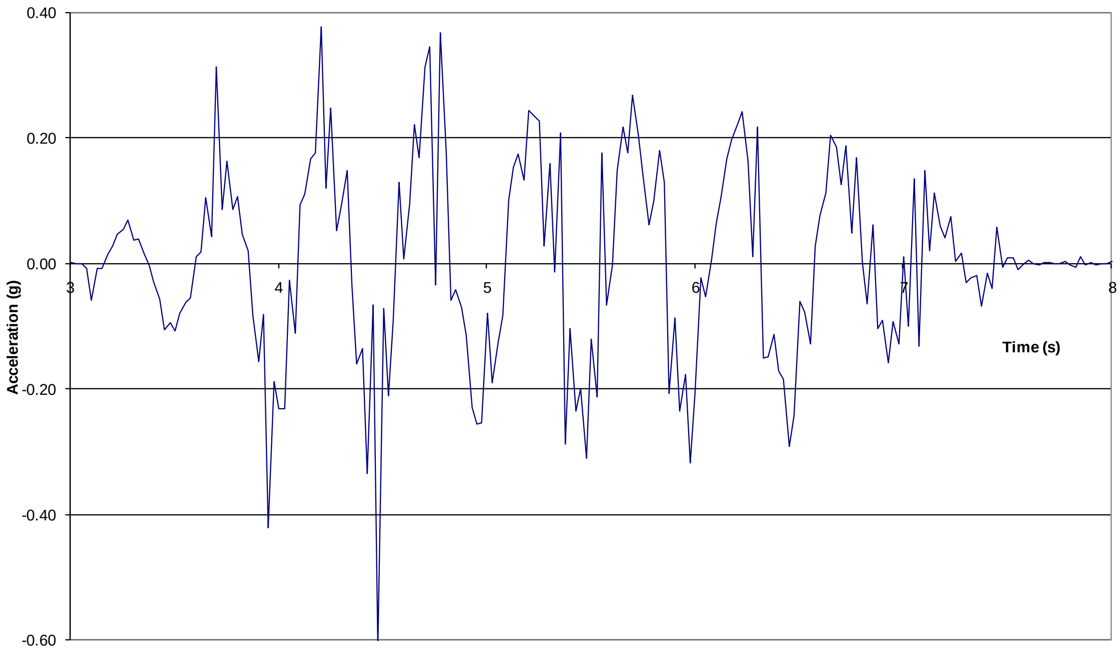


Figure 28. Measured table acceleration in Test R16

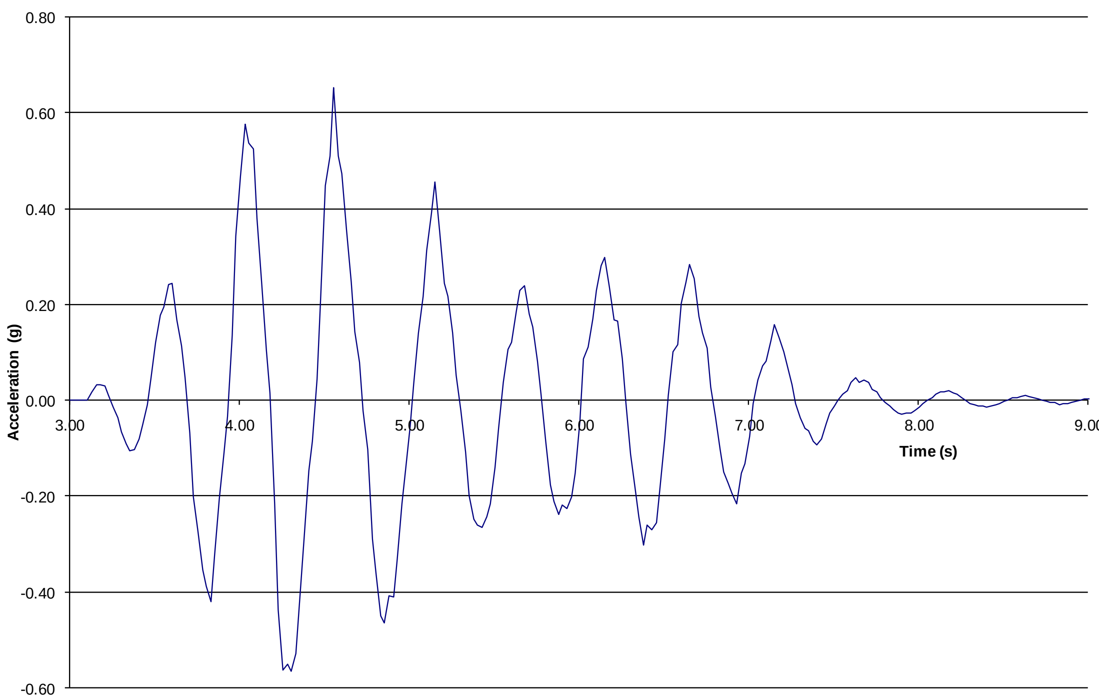


Figure 29. Measured ceiling acceleration in Test R16

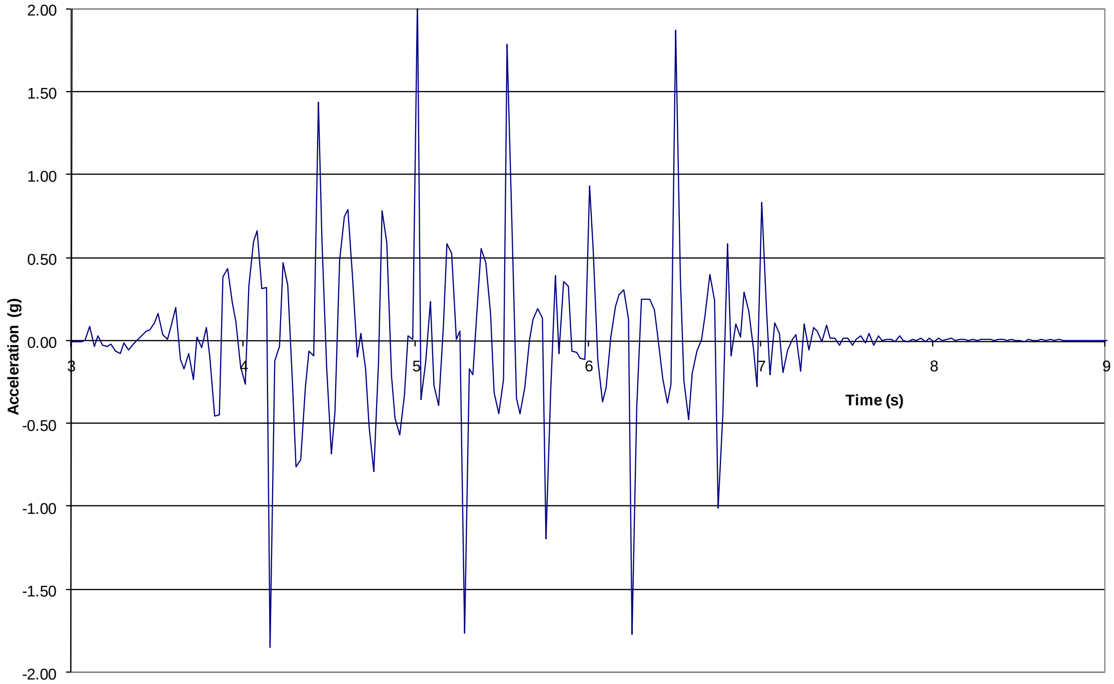


Figure 30. Measured Side 1 top of veneer acceleration in Test R16

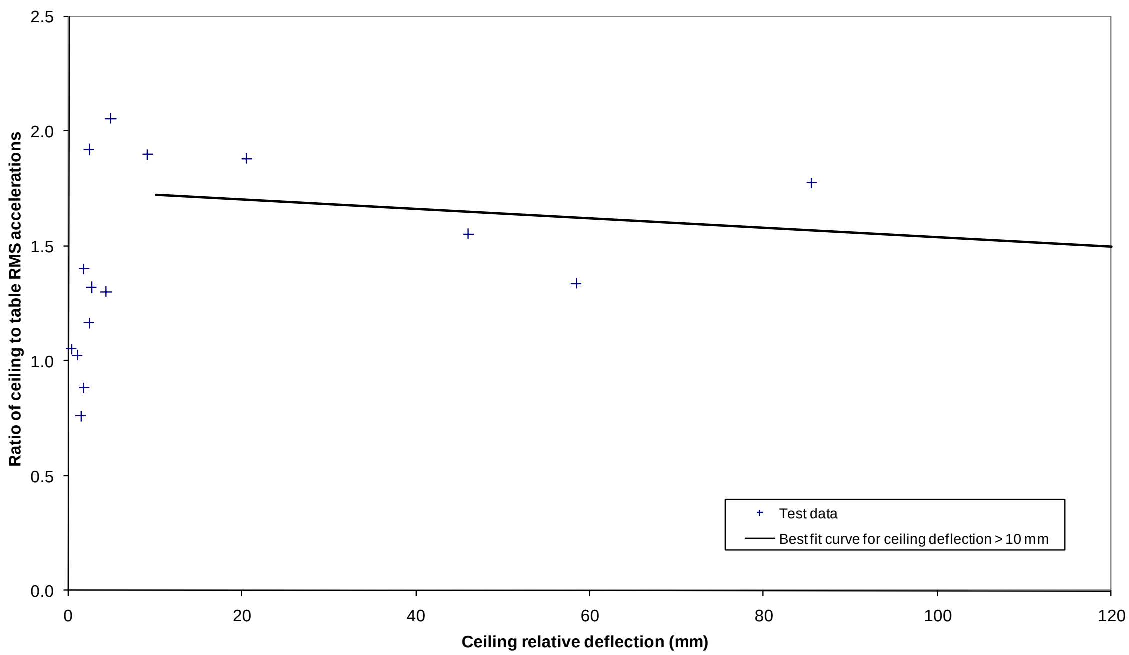


Figure 31. Ratio of ceiling RMS acceleration to table RMS acceleration versus ceiling deflection

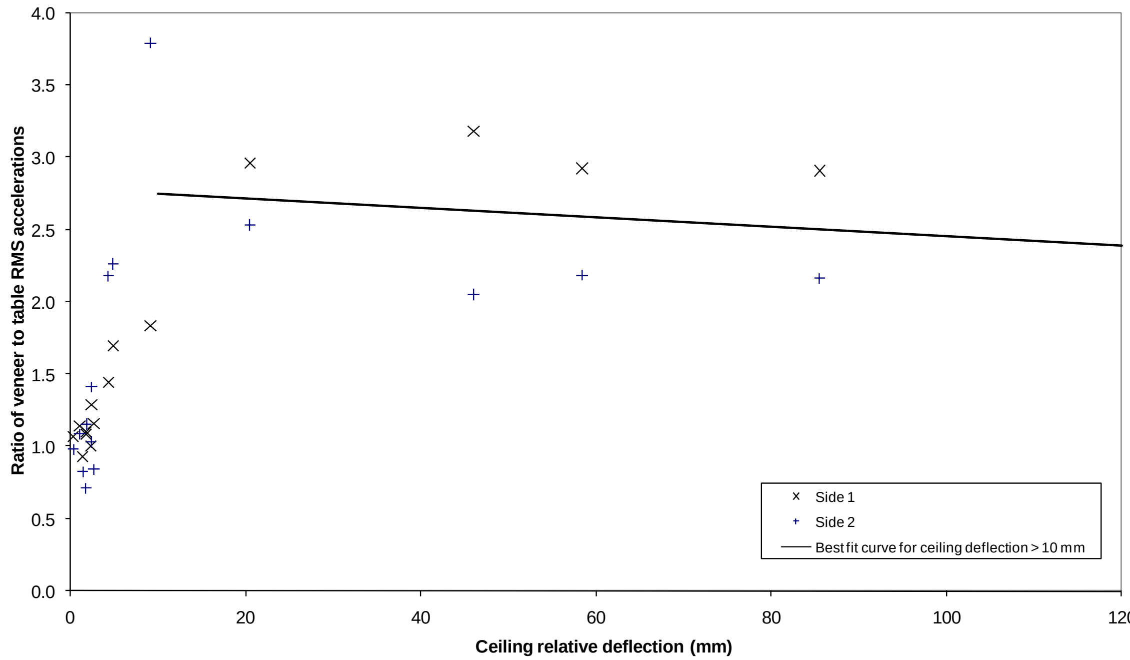


Figure 32. Ratio of veneer RMS acceleration to table RMS acceleration versus ceiling deflection

9. RESPONSE SPECTRUM

In all shake table tests the table accelerations were used to generate elastic acceleration response spectra for 5%, 10% and 20% damping. A typical example (taken for Test R16) is shown in Figure 33. The excitation frequency in this test was 2 Hz (period = 0.5 seconds). The ordinate at the peak for the 5% damping curve is 2.10g. Thus, a single degree of freedom (SDOF) elastic oscillator, carrying a weight of 10 kN, with natural period of precisely 0.5 seconds and damping of 5%, will have a peak base shear force of 10 x 2.10 = 21 kN when subjected to measured table accelerations.

Figure 34 shows the total response spectra obtained by accumulating all the shaking up to the end of Test R14 (i.e. while the lining fixing was still effective and deflections did not exceed 10 mm). Also plotted in Figure 34 are the NZS 4203 elastic design spectra for Wellington factored by 1 and 2. In Section 7.1 it was shown that the room natural period for this level of shaking when the room lining was still effective, was between 4 and 5 Hz (i.e. 0.25 to 0.20 seconds). Considering this zone of Figure 34 it can be seen that the shaking imposed corresponds to approximately 1.0 times the design spectra. Note that the lining was only lightly fixed in the LTF and thus much of the lateral strength may have been provided by the veneer.

Also shown in Figure 34 is the sum of the response spectra applied to the clay brick veneer room (Thurston and Beattie 2008b) for the shaking where the lining fixings were still effective and the deflections did not exceed 10 mm. The clay brick spectra are approximately 70% greater than the concrete brick in the 4 to 5 Hz zone.

The lining lost most of its racking effectiveness in Test R15 at 3.0 Hz shaking, which induced ceiling relative deflections of ±20.5 mm. The deflections reached ±46, 58 and 86 mm in the subsequent tests R16, R17 and R18 at 2.0, 1.5 and 1.0 Hz respectively. The sum of the response spectra for these tests where the lining had become ineffective is given in Figure 35. In Section 7.1 it was shown that the room natural frequency when the room lining had become ineffective was less than 1.5 Hz and probably close to 1.0 Hz (i.e. a period of 0.67 to 1.0 seconds). Furthermore, it was concluded that the response was not particularly sensitive to the shaking frequency in this range. Thus, considering the portion of the plots in Figure 35 in the frequency range between 1.0 and 1.5 Hz, the maximum shaking is considered to be approximately 1.4 times the NZS 4203 design spectra.

Also shown in Figure 35 is the sum of the response spectra applied to the clay brick veneer (Thurston and Beattie 2008b) in tests T24 to T31 and T17 to T21 (i.e. after the LTF deflections had exceeded 17 mm and the lining fixings had therefore become ineffective). These spectra are approximately 140% greater in the 1.0 to 1.5 Hz zone. Hence, the concrete brick veneer experienced significantly more damage at much lower shaking intensity than the clay brick veneer.

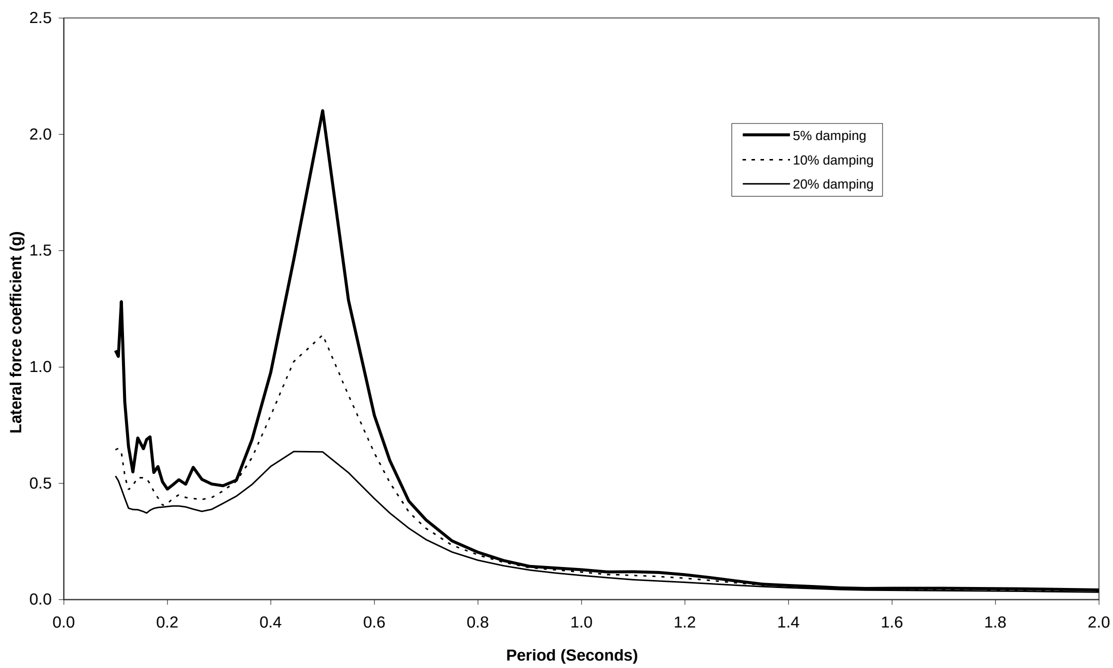


Figure 33. Response spectra from shaking of Test R16 at 2 Hz

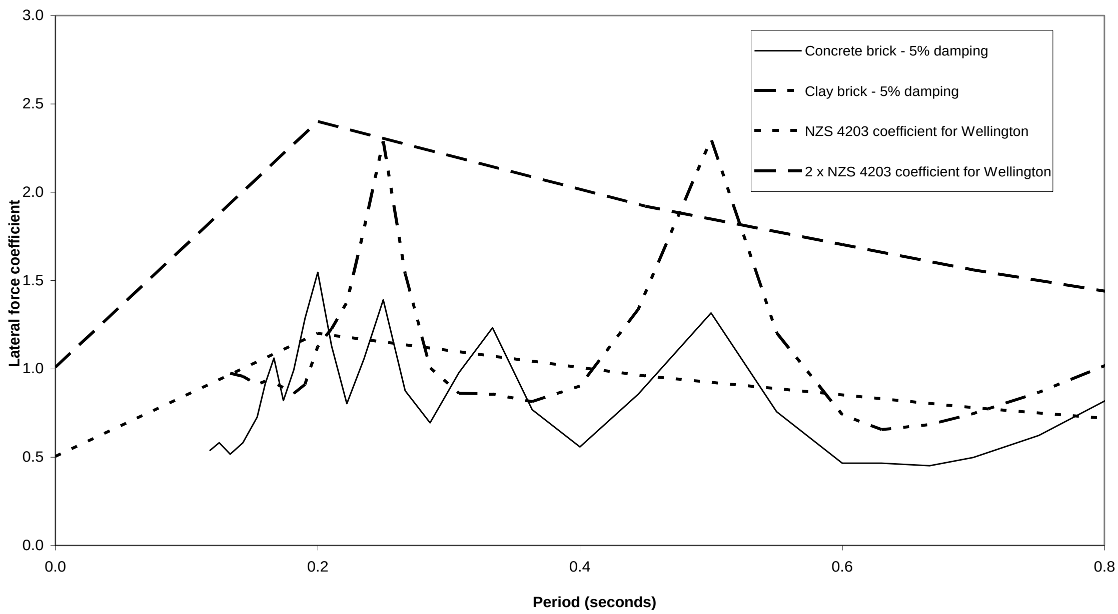


Figure 34. Summation of response spectra before LTF deflections had exceeded 10 mm

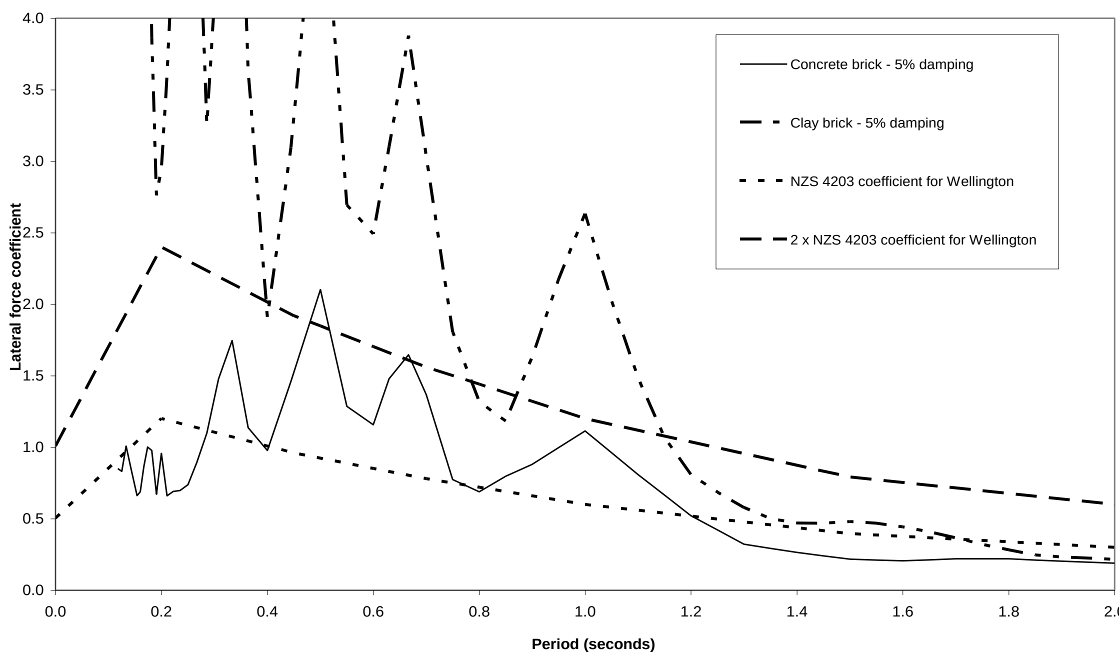


Figure 35. Summation of response spectra after LTF deflections had exceeded 10 mm

10. MAGNITUDE OF LOADING

It is important to assess the degree of shaking imposed in the shake tests on the room and to relate this to the design level of shaking for such construction. That is, the shaking needs to be related to the design strength of the LTF bracing walls, the NZS 3604 demand load for the mass used, the recorded table accelerations and the NZS 4203 design spectra.

From the report for the clay brick veneer shake testing (Thurston and Beattie 2008b) the effective building weight at ceiling level was:

Source of weight	kN
Added weight on top of ceiling	34.9
Half the weight of the concrete veneer	17.9
Ceiling weight	2.6
Half the weight of the LTF walls	1.5
	56.9

The added weight on top of the ceiling corresponded to 6% more than the estimated weight per unit length of a 10 m wide house clad with veneer and with a heavy roof.

Shelton (2007) stated that the NZS 3604 seismic design force at a particular level for Zone A was 0.241 x the weight at that level. Hence, the NZS 3604 seismic design force = 0.241 x 56.9 = 13.7 kN.

From Thurston and Beattie (2008b) the design strength of the plasterboard-lined LTF was 10 kN. Hence, the design seismic load was 13.7/10 = 1.37 times the resistance provided if the bracing strength of the veneer is ignored.

In Section 9 it was shown that, based on the measured accelerations, the shaking imposed was approximately 1.0 times the design shaking at the time of lining failure and 1.4 times at veneer failure. The design loading can be found by factoring these values by 1.37, giving rounded values of 1.4 and 2.0. Thus, the imposed loading was approximately 1.4 times the design loading at the time of lining failure and 2.0 times at veneer failure.

Thus, good performance can be expected for this type of construction in a design level earthquake. However, failure is likely to occur at earthquake load levels above this value.

11. SUMMARY

A concrete brick veneer room was subjected to sinusoidal shaking. When the deflections became large the LTF lining became ineffective as a bracing element and eventually fell off. The veneer panels, which were loaded in-plane, took up most of the ceiling movement by rocking. The face-loaded portion of the veneer stayed intact and moved with the end LTF walls. Near the end of the testing a few brick ties developed fatigue cracks and some of these failed in the last test, but this was attributed to the large number of cycles imposed on the test room and is unlikely to occur in practice from earthquake loading. All the ties remained fully embedded in the mortar and attached to the LTF framing with no slip being observed.

At a shaking level of approximately 1.4 times the loading expected in a design earthquake, the LTF room deflection was less than ±10 mm, even though the lining had been only lightly attached to the LTF and would have therefore contributed only a low bracing resistance. During the shaking the veneer resisted a significant portion of the imposed lateral forces. During this testing, the veneer panel movements were almost entirely accommodated by rocking, with no slippage along mortar joints. After the imposed shaking was stopped, the veneer dragged the LTF back to close to the initial zero position, veneer cracks closed and little damage to the veneer was visible. At this stage, simply re-pointing the cracks was expected to be a suitable method of repairing the veneer.

At a shaking of approximately twice the loading expected in a design earthquake, the LTF ceiling deflection had increased to ±86 mm. Slippage along the veneer horizontal mortar joints was large. The veneer panels had begun to disintegrate into separated bricks and some bricks fell at corners and at the edges of openings. Damage at veneer corners was particularly severe. A veneer in this condition following an earthquake is expected to be demolished rather than repaired.

12. CONCLUSIONS

Two brick veneer rooms have been tested. The results of testing the first of these, using a clay brick veneer, have been reported in Thurston and Beattie (2008b) and the results of the tests on a concrete brick veneer room have been reported here. The clay bricks had internal holes into which mortar penetrated in the normal brick-laying operation and these formed mortar dowels which tended to hold brick layers together. The concrete veneer bricks were solid, thus preventing any mechanical interlock from the mortar. Based on the measured accelerations, the clay brick was subjected to 140% greater seismic loading but (apart from damage to the brick ties) experienced significantly less damage than the concrete brick veneer did.

More ties suffered fatigue cracking and failure in the clay brick veneer tests than occurred in the concrete brick tests. This was attributed to the clay brick veneer being subjected to many more cycles to large deflections. However, fatigue cracking and failure of the brick ties is considered to be unlikely in houses designed to the current NZS 3604 procedure or the proposed new procedure described in Thurston and Beattie (2008b), even for maximum credible seismic events. No ties disconnected from the LTF or pulled out of the brick mortar in the clay or concrete brick veneer tests.

At 1.4 times the design earthquake level the concrete brick veneer room behaved well and the veneer did not break up, could be easily repaired and provided both bracing and stiffening to the room. However, at approximately twice the design earthquake level the concrete brick veneer separated into individual bricks, especially at corners and the edges of openings, with some bricks falling. In the latter stages of the tests on the concrete brick veneer, as the individual bricks separated from adjacent bricks, the ties did pull out of the veneer. The veneer would have needed to be demolished if this had occurred in a real building. To avoid this undesirable behaviour, it is recommended that the bracing strength of veneers made of bricks without holes not be used in design, such as is recommended for veneers made with bricks containing holes. (Thurston and Beattie 2008b). The current design procedure (where no account is taken of the bracing ability of the veneer) is expected to generally result in good performance in buildings using this type of construction, when subjected to design level shaking. A large earthquake occurred in Gisborne while this investigation was being undertaken and a concrete brick veneer house suffered significant damage. However, the house was under construction and there were no interior linings in place. There were some diagonal braces present but most of the seismic load had to be resisted directly by the veneer. It is also suspected that the age of the mortar was short and it may not have reached its design strength.

Brick veneer houses with significant plasterboard bracing provided by the lining will probably have a natural frequency greater than 5 Hz. If the building is founded on hard or intermediate soil the high frequency components of a large earthquake may well crack the veneer and cause some lining damage. However, it is unlikely to also have sufficient low frequency components in the 1-2 Hz range to make the veneer rock to large deflections. If the building is founded on soft soil it is unlikely to have sufficient high frequency component to induce lining damage. Hence, the combination of lining and veneer bracing offers good seismic protection for the concrete veneer construction tested in this study.

The test room veneer behaviour was consistent with what was observed in the slow cyclic tests reported by Thurston and Beattie (2008a), namely:

(1) The end wall ties prevented the L-shaped corner veneer elements from slipping along the base. Thus, they simply rocked with very little horizontal distortion of the ties (generally less than 10 mm). Significantly greater tie distortion occurred in isolated in-plane loaded brick veneer piers.

(2) There was little out-of-plane differential movement between the end veneer and LTF walls. The small amount that did occur (generally less than 10 mm) was due to compressive buckling of the ties. This movement was similar to the in-plane movement which occurred in L-shaped corner veneer elements as noted in (1) above.

13. REFERENCES

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