STUDY REPORT

No. 190 (2008)

Seismic performance of brick veneer houses

Phase 2. Shake table tests on a clay brick veneer specimen

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Preface

This is the third 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 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-framed (LTF) construction should be designed to carry the entire load.
- To identify the 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. MonierBrick donated the bricks and Eagle Wire Products Ltd donated the brick-ties used in the testing.

Note

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

SEISMIC PERFORMANCE OF BRICK VENEER HOUSES.
PHASE 2. SHAKE TABLE TESTS ON A CLAY BRICK VENEER SPECIMEN

BRANZ Study Report SR190 (2008) **S.J. Thurston and G.J. Beattie**

Reference

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Abstract

Historically brick veneer houses have not performed well in earthquakes. However, 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) is expected to result in a better performance.

BRANZ performed sinusoidal shake table tests on a large 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 related to the design earthquake by:

1) comparing the design spectra with that calculated from measured table accelerations; and
2) the brick veneer lateral strength calculated from measured tie deformation properties and veneer weight.

Despite the large number of cycles imposed (including 40 cycles at 3.0 times the design load) and LTF wall deflections reaching ±118 mm, the veneer returned to close to its original position and veneer cracks largely closed and could be repaired by re-pointing.

It was concluded that properly constructed brick veneer on all sides of a house which incorporated resilient ties and had bricks with vertical holes could be relied upon to resist design earthquake events with low building deflections. The veneer would be easily repairable. Collapse would be unlikely under shaking several times the design event.

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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 current 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.

1.2 New Zealand building standards

Since the early 1990s changes have been instigated to improve the seismic performance of brick veneer, particularly under face loading. Shelton (1996) investigated the face-load resistance of brick veneer ties and determined that for satisfactory out-of-plane performance of the veneer it was necessary to screw-fix the ties to the timber framing. This new requirement was incorporated in the construction requirements for brick veneers in NZS 4210:2001 (SNZ 2001). This standard includes a requirement that the ties are to be screw-fixed to the studs and that the tie-to-brick joints have sufficient strength and stiffness. It also requires the ties to be fully encapsulated in the mortar. There is therefore a mix of prescriptive and performance requirements included in the standard. It has since been found that ties may be dry-bedded and still satisfy the performance requirements of the standard for ties under face loading (Beattie 2006). No requirements are contained in the New Zealand standards for the in-plane performance of the ties. AS/NZS 2699.1 (SA/SNZ 2000) provides a test procedure to establish the rating of the veneer ties in terms of their axial stiffness and strength. The test method notes that when cycling the ties horizontally in the wall plane direction, "designers may find the loads resisted and stiffness encountered during this regime useful" and it suggests that manufacturers collect this information. However, this has not generally been done by any testing agencies.

1.3 Previous investigations

Literature searches have shown that little work has been done either in New Zealand or overseas to understand the in-plane performance of the combination of veneer and framing, particularly at wall corners (Beattie 2006). Beattie found that research had been undertaken on the in-plane performance of veneers by Lapish and Allen in the late 1980s (Lapish 1991, Allen and Lapish unknown date). Lapish noted that "because of the rigid, brittle nature of unreinforced masonry veneers and the limited ductile nature of reinforced veneers, the in-plane seismic design loads generated within the mass of the veneer must be at a high level to assume full or near-elastic response to the earthquake forces". The report went on to state that the veneer must have a separation from the flexible seismic load-resisting structure to prevent altering the intended seismic response.

Allen and Lapish described full-scale in-plane racking tests. The panels were approximately 2.4 m high by 2.4 m long. Load was applied to the top plate of the timber-framed wall, and during the tests the veneer panel was observed to lift off the foundation as it was pulled by the ties connecting it to the timber frame. (Similar behaviour was observed in research undertaken by Beattie (2003).) No distress was observed in the brickwork. Allen and Lapish concluded that:

1. The brick panels of these dimensions were able to sustain lateral deflections up to ±25 mm without any real damage to the face-fixing capacity of the stiff strip metal ties, and the brick panel rocked in concert with the racked framing.
2. The design of timber-framed buildings with stiff wall tie anchorage into brick veneers should take account of higher earthquake loads than for those situations where the veneer connectors do not materially influence the racking capability of the timber-framed walls.
3. Conventional stiff masonry ties should be used only where veneers are designed and detailed into individual panels which are free to rock as units under lateral loads.
4. Stiff brick veneer ties should be ductile.

Johnson and McGinley (2003) conducted in-plane shear tests on small (900 x 600 mm) veneer panels connected to timber stud wall sections with corrugated metal ties. The tests were monotonic. In their Introduction they note that the 2000 International Residential Code (ICBO 2000) restricts the height of masonry veneer that can be used in higher seismic zones to one-storey structures. They note that "there may be a substantial in-plane shear transfer between the wood frame and the brick veneer…." resulting in the participation of the veneer in the lateral load-resisting system. They go on to show that there is sufficient strength to resist significant wind loads, but there is no mention of cyclic response in earthquakes. While the authors refer to whole house testing in Australia and Japan, their reference list does not include relevant papers from either of these countries.

Choi and LaFave (2004) carried out cyclic in-plane tests on brick couplets fixed to timber stud members with 22g, 28g and 16g corrugated metal ties. The ties were nailed into the stud elements. Strengths were reported for the three gauges of metal, but little information was provided on the large displacement performance.

Heath et al (2006) conducted in-plane shear tests on brick veneer panels 3 m long by 2.4 m high with an included window opening. Because they were interested in the development of cracks in the veneer due to underground blasting, they carried out a monotonic racking test in accordance with ASTM Standard E72 (2005) after cutting a significant proportion of the ties joining the veneer to the framing. In-plane load was applied directly to the top of the veneer and the end of the veneer was restrained vertically. Cracks were observed to develop at the corners of the window opening. These began to form at a top displacement of 12 mm and continued to form up to 30 mm. However, because there was no interest in interaction between the veneer and the framing, no comment was made on this in the paper.

Heath et al (2008) undertook shake table testing on a 2.6 m by 2.8 m room with brick veneer cladding to simulate ground vibrations from blasting. The specimen included window and door openings but the veneer panels on the four sides were not connected together at the corners. While the development of cracking in the individual panels was closely monitored as the excitation was increased, there was no cracking at the corners related to interaction of the face-loaded and in-plane-loaded veneer elements.

While the aim has been to maintain the link between the veneer and the frame under face loading, the effectiveness of these measures had not been proven in any reasonable scale investigations.

1.4 BRANZ investigations

BRANZ undertook an investigation to determine the in-plane shear strength of two brick ties typically used in New Zealand (Beattie 2007). The results of these tests showed that there was both a significant transfer of in-plane forces between the timber frame and the veneer and that the veneer ties were able to accommodate large differential in-plane displacements between the veneer and the framing while transferring this load.

BRANZ has also recently undertaken slow cyclic racking tests on two specimens with brick veneer cladding. The results of this study are reported by Thurston and Beattie (2008a). This report should be read in conjunction with this report.

1.5 Investigations in USA

Through contacts (Shing 2008), BRANZ is aware of a research programme being undertaken in the USA. The programme is concerned with the performance of reinforced masonry walls, but a part is concerned with the seismic performance of brick veneer attached to timber framing with American style ties. These ties are considerably weaker and less flexible than the ties used in New Zealand. A one-directional shake table test is planned on a 6 x 6 m single-storey structure in January 2009.

1.6 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 performed 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 is also likely to 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 connection improves the sliding resistance of the side veneer panels. The rocking behaviour of the L-shaped corner veneer elements would also be affected if cracking occurs in the corners.

1.7 New Zealand brick veneer construction

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”.

1.8 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 clay bricks used had dimensions 230 mm long x 76 high x 70 mm wide. When assembled using the standard 10 mm of mortar between bricks, the veneer weighs approximately 130 kg/m2.

The bricks used had five vertical holes, of cross-section 32 x 23 mm, for the full brick depth which partially filled with mortar as shown in Figure 2. This effectively formed mortar dowels which greatly enhances the horizontal shear strength between bricks. Thus, in the tests described in this report, brickwork did not slide along horizontal mortar cracks between bricks but isolated veneer panels did slide on the smooth face of the foundation concrete.

The bricks were laid by tradespeople using Dricon® Trade Mortar with approximately 10 mm thick mortar beds being used between the bricks on both horizontal and vertical surfaces. This mortar was stated to comply with NZS 4210 (SNZ 2001) for masonry construction. Compression test cylinders were cast from the mortar mix to determine the compressive strength of the mortar.

Hot-dipped galvanised, 85 mm long, 70 series, Eagle brand brick-ties were dry-bedded onto the bricks rather than being fully encapsulated within the mortar. The ties were stated to be rated “heavy earthquake to NZS 3604:1999 and the draft AS/NZS 2699 standards”.

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.

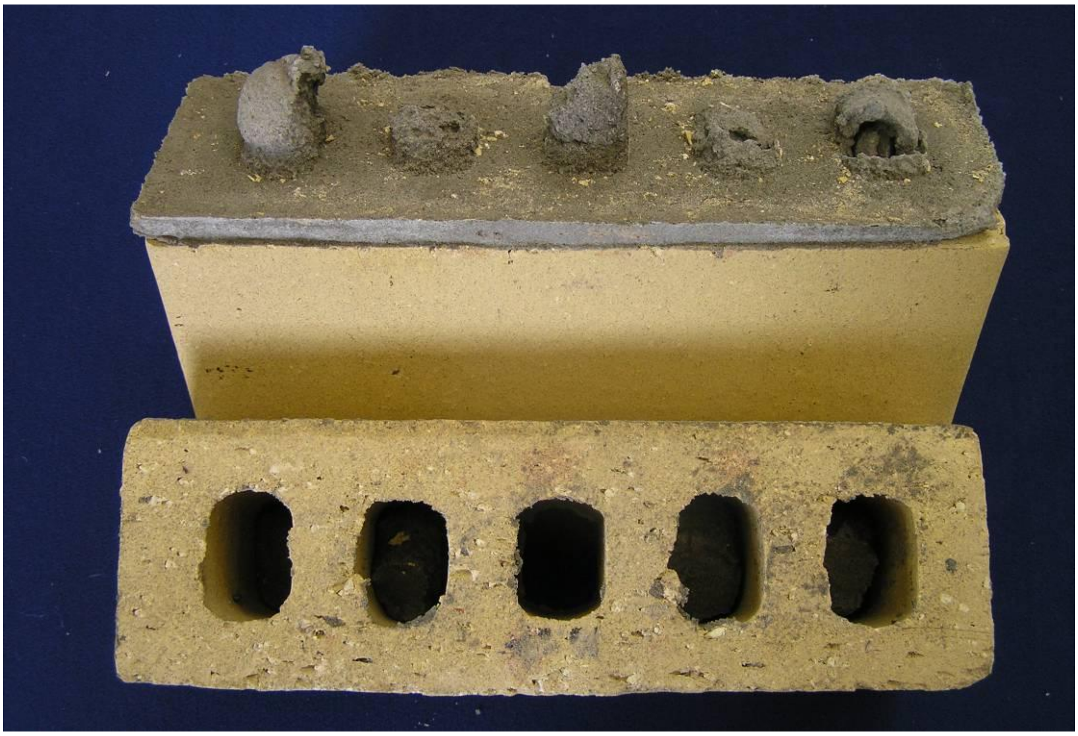


Figure 2. Typical 70 series clay brick – note how the holes in the bricks allow mortar dowels to form

1.9 Limitations of this study

This study only considers houses with clay brick veneer around the complete house perimeter and the modern construction practice of screw-fixed ties. The report is also only concerned with modern 70 series clay brick veneer construction where penetrated bricks, such as shown in Figure 2, are used. The findings are particularly not applicable to older veneers of approximately 100 mm thickness and constructed with solid bricks.

1.10 Outline of report

This report describes the construction and instrumentation of a full-scale 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, and then it considers the application of a computer model to define the behaviour of brick veneer under seismic loading.

2. CONSTRUCTION OF TEST ROOM

The test room was a single-storey nominally 2.4 m high room of dimensions 4.1 m long (in the direction of shaking) by 2.5 m wide. It had plasterboard-lined LTF walls, brick veneer on all four sides and a timber-framed plasterboard-lined ceiling. Figure 3 shows a plan cross-section of the test room and 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 4 and Figure 5 are elevations of the LTF walls showing framing and the added weights to simulate the inertial mass of the roof. Figure 6 and Figure 7 are photographs showing the veneer on the two side walls and some of the instrumentation is also identifiable. Figure 8 shows a cross-section through the base of the walls. The veneer was spaced from the LTF by 50 mm. It is a requirement of NZS 3604 (SNZ 1999) that the cavity is between 40 and 75 mm wide. 50 mm is commonly used by brick layers to keep within the tolerances.

As shown in Figure 3, 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 i.e. with no end restraint. This 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 panels between windows to be incorporated – referred to as "piers".

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, in Stage I of the testing the lining was only lightly attached to the framing as shown in Figure 9(a). The lining was replaced in Stage II and the more dense screw-fixing pattern of Figure 9(b) was used. The reasons for the chosen lining strength are discussed in Section 4. 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 tabulated below.

2.1 General

- The outside plan dimensions of the brick veneer were 4.07 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 26 courses of clay bricks placed in running bond. Galvanised steel ties were placed on top of the 2nd, 6th, 10th, 14th, 18th, 22nd and 24th 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 10.

Details of the bricks, brick-ties and mortar are presented in Section 1.8.

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 8. The steel channel was rigidly attached to the top of the shake table.

The brick veneer was constructed on a steel channel filled with concrete to provide a bed for the brick veneer, as shown in Figure 8.

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 3 and Figure 12. Note that care was taken to ensure these measurements included all crack openings at these plan locations.
- 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 6.
- The in-plane horizontal deflection near the bottom of the brick veneer relative to the shake table at mid-length of each side panel as shown in Figure 11.
- The horizontal deflection of the ceiling relative to the shake table at the mid-width of the room. This was achieved by constructing a 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.

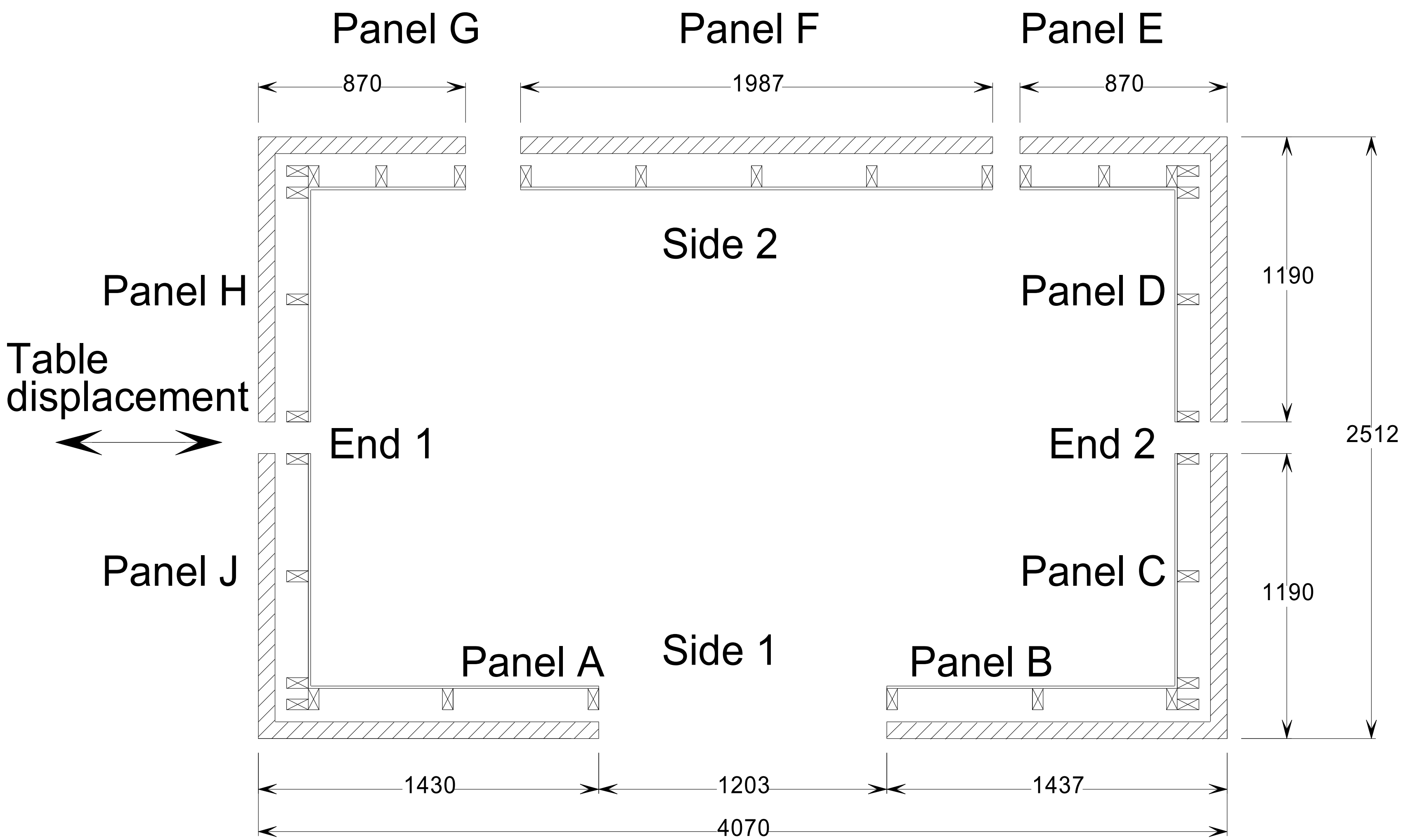


Figure 3. Plan section showing dimensions and labelling system

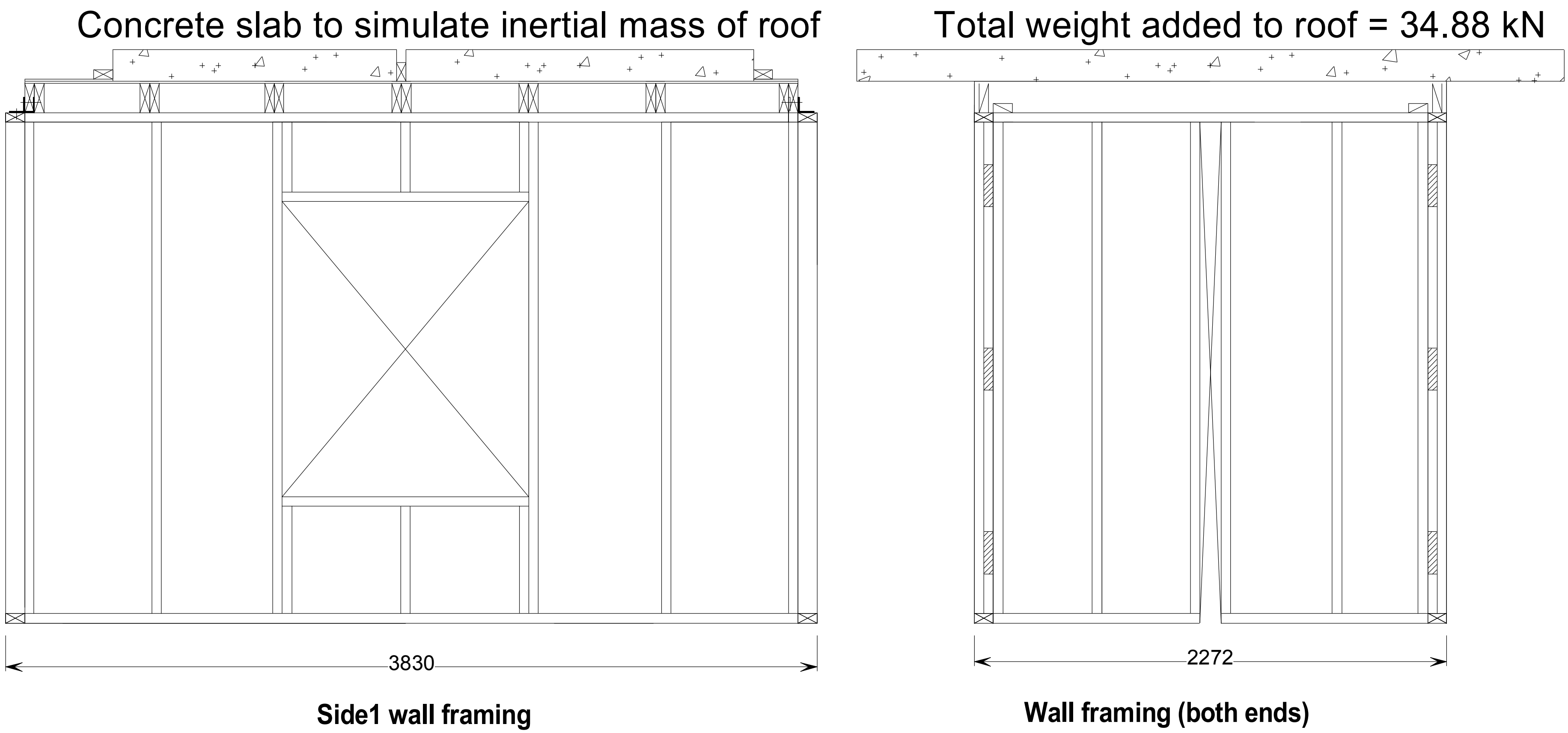


Figure 4. LTF framing on Side 1 and the end walls and added roof weights

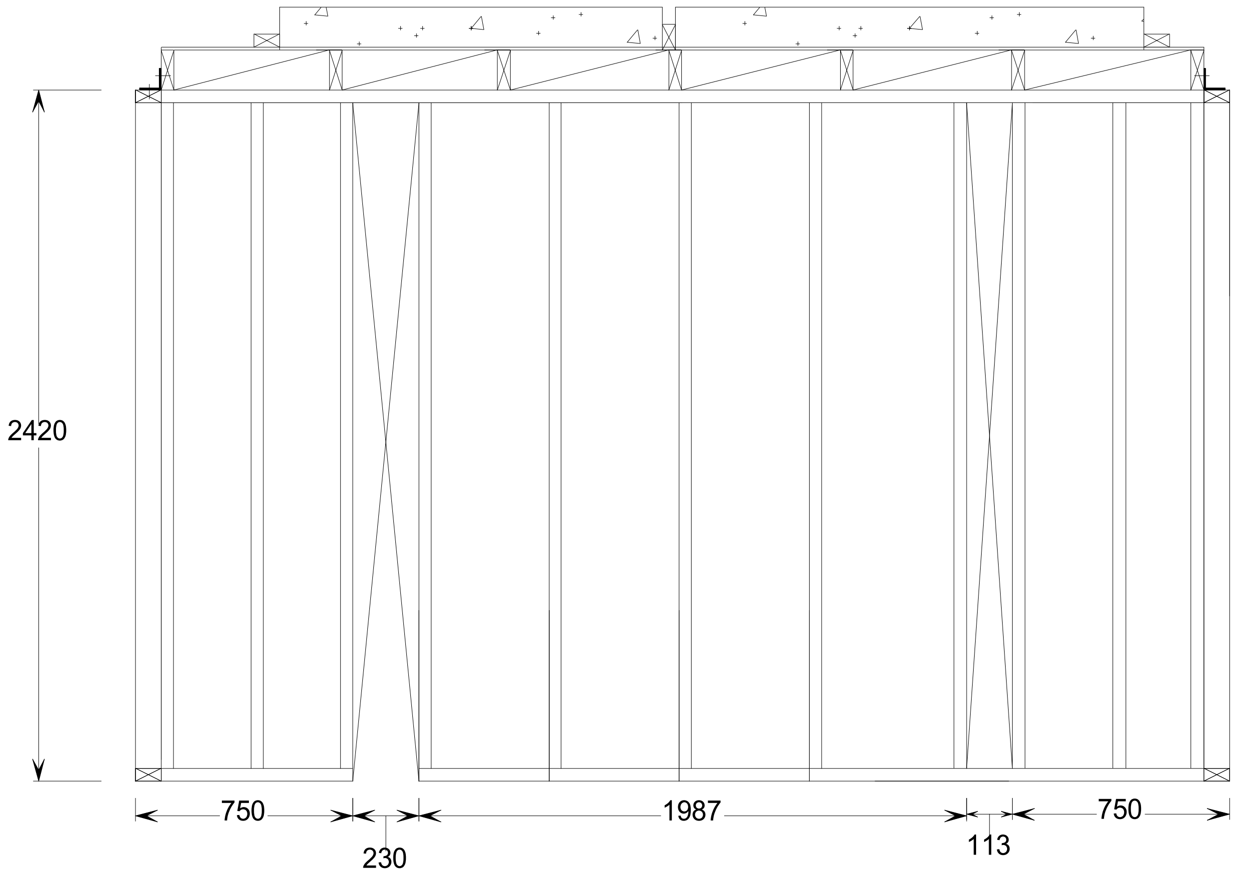


Figure 5. LTF framing on Side 2

Figure 6. View of Side 1 before testing

Figure 7. View of Side 2 in the early stages of testing

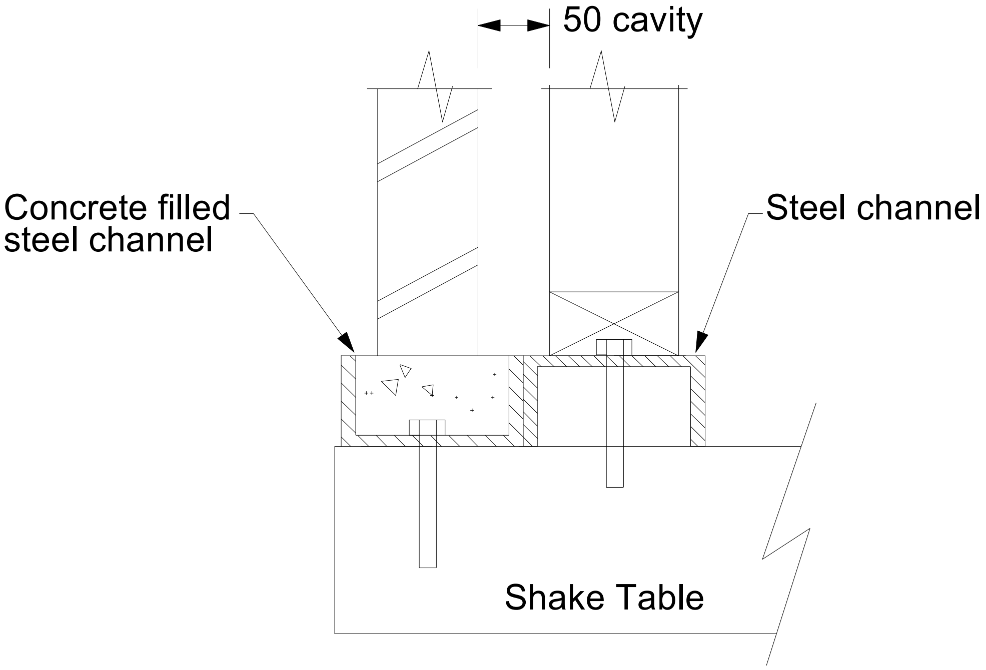
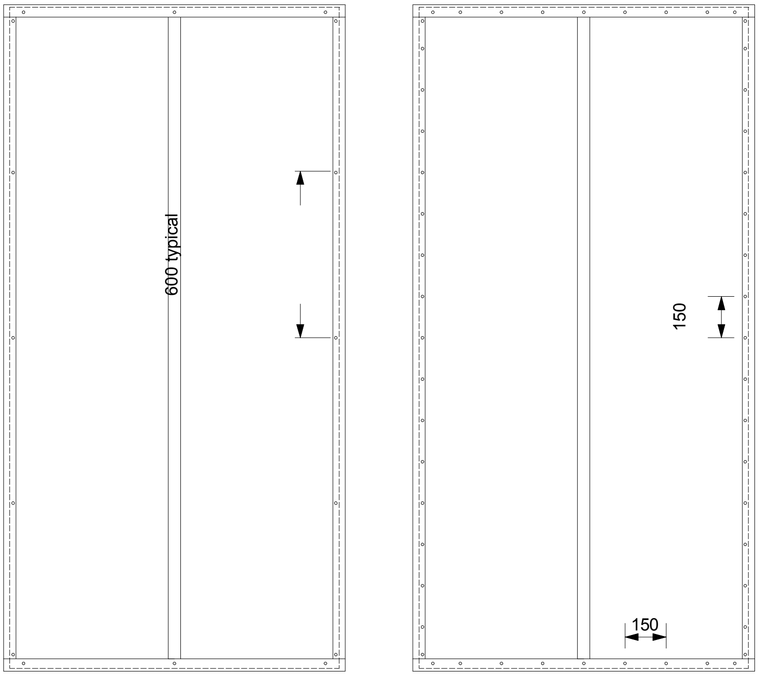


Figure 8. Cross-section through bottom of veneer and LTF walls

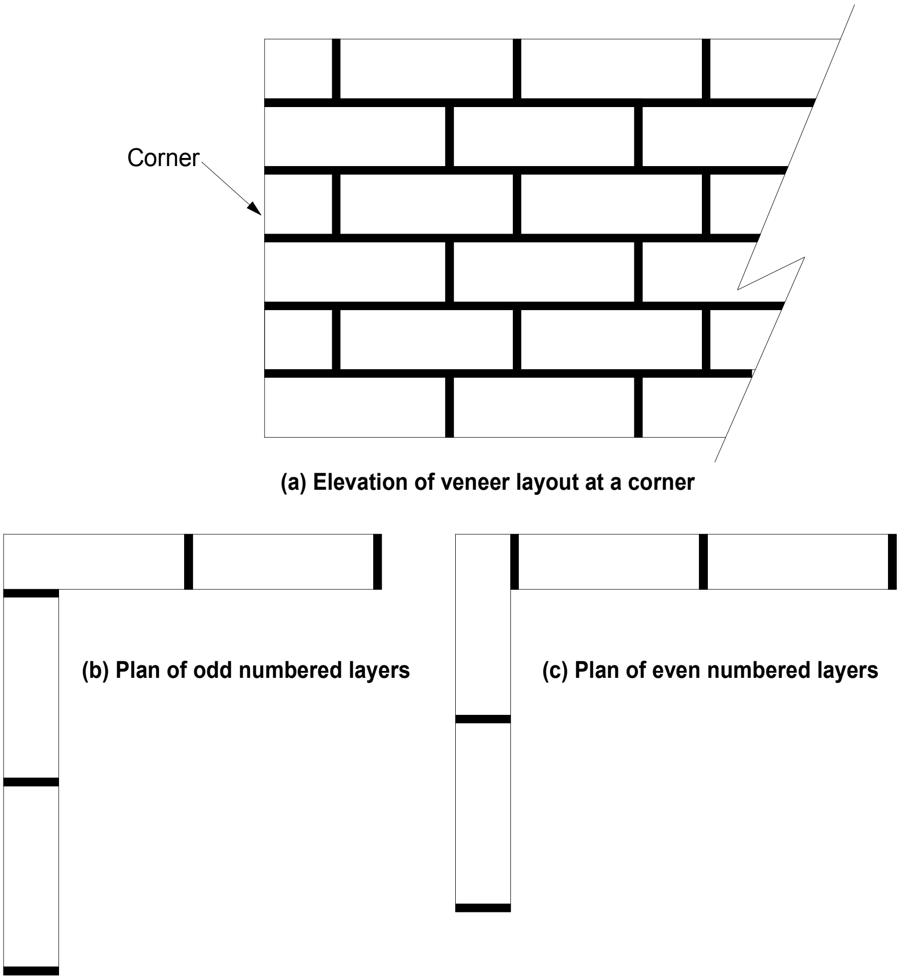


(a) Fixing for Stage I.
Screws at 600 mm centres

(b) Fixing for Stage II.
Screws at 150 mm centres

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

Figure 9. 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 10. Interlocking of brick veneer at corners

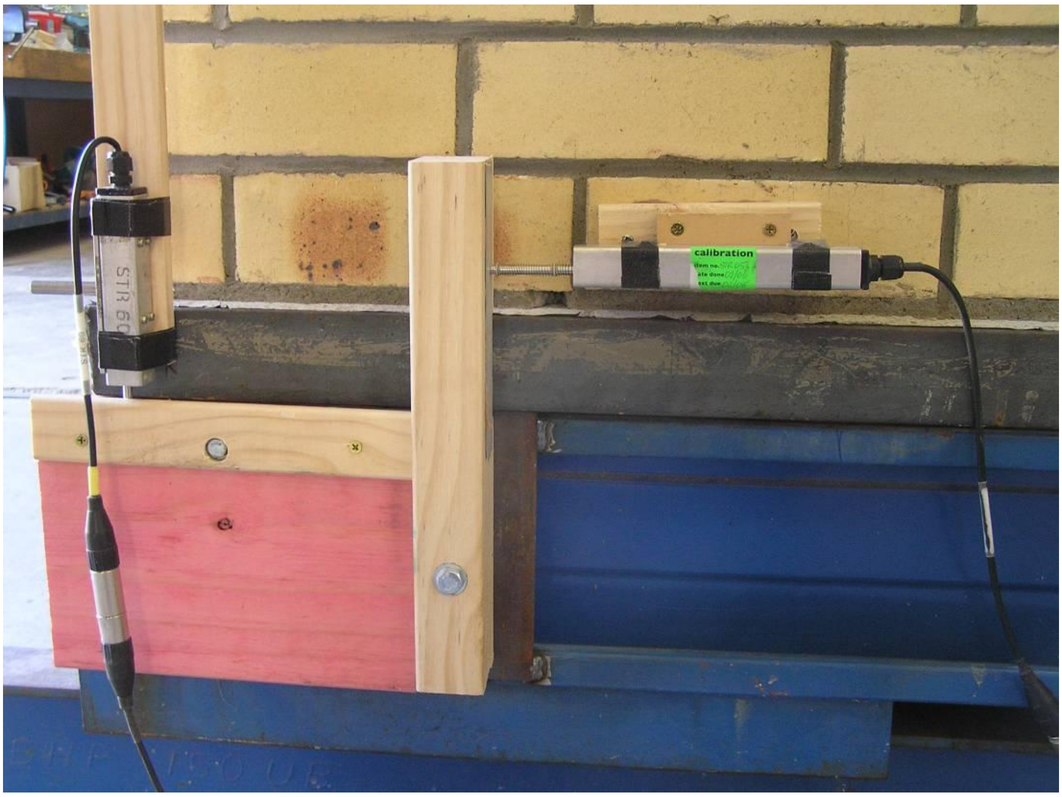


Figure 11. Gauges measuring veneer vertical and horizontal movement


Figure 12. Gauges measuring veneer vertical movement

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

4.1 Room weights

The LTF framing had an overall length and width of 3.83 m and 2.272 m, respectively. The room was intended to simulate a 3.83 m long portion of a 10 m wide house with a heavy roof of weight 0.6 kPa and ceiling of weight 0.24 kPa, which are what is assumed by NZS 3604 (SNZ 1999) in the calculation of bracing demand. If the eaves are 0.6 m wide, then the weight of the roof and ceiling of a 3.83 m long portion of the house W_c, was given by:

W_h = 3.83 x {(10+2x0.6) x 0.6 + 10 x 0.24} = 34.93 kN

The as-built shake table room ceiling weight, W_c, was estimated to be:

W_c = 3.83 x 2.272 x 0.24 kPa = 2.08 kN

Hence, the weight required to be added to the roof of the shake table room was:

$W_h - W_c = 32.85$ kN.

The measured weight of the added concrete slabs actually was 34.88 kN, which is only 6% greater than required by the above calculation.

4.2 Shake table limitations

The capacity of the servo-valves and pump limited the shake table to a maximum velocity of approximately 250 mm/second if at low load. The maximum achievable velocity drops at high load. Thus, using the simple harmonic motion formulae, the acceleration was limited to ((250 x 2πf)/9806)g (= 0.16g at 1 Hz, 0.32g at 2 Hz, 0.48g at 3 Hz, 0.64g at 4 Hz), although the peak accelerations at higher frequencies were unlikely to be achieved due to pump limitations. (Actual values achieved in the test were 0.16, 0.31, 0.42, 0.5 and 0.77g at 1, 2, 3, 4 and 9 Hz, respectively.)

4.3 Predicted LTF wall and veneer racking strengths

There was a concern that the shake table would be unable to induce failure in the test room. For a room lined with Multiboard plasterboard fixed at 150 centres (but without joints reinforced with paper tape and stopped), the estimated strength of the LTF lining was 28 kN which would peak in strength at 12–16 mm racking deflection and then degrade moderately rapidly. Using the theory by Thurston and Beattie (2008a) the predicted strength of the brick veneer (see Section 11) was 21.4 kN at an LTF deflection of 10 mm and this would remain fairly constant with greater LTF deflections. This gave a total strength of 28+21.4 = 49.4 kN. This implied that the test room effective mass at ceiling level of 58.3 kN (see Section 11.1) had to be shaken at 49.4/58.3 = 0.85g. If a dynamic amplification factor of 2 was assumed the table acceleration would have to be approximately 0.43g. As there was doubt that this could be achieved the following construction was used:

- In Stage I, only a weak lining fixing was used, as shown in Figure 9(a). The calculated LTF bracing strength for the entire room using this lining, based on wall bracing tests, was calculated to be 10 kN.
- In Stage II, a more dense lining fixing pattern was used, as shown in Figure 9(b), and the LTF bracing strength was calculated to be 28 kN.

The above values were confirmed, within 10%, by racking tests on the room with newly lined walls. Note, that the vertical joints between sheets were not reinforced with paper tape and stopped as this would have resulted in significantly increased LTF wall bracing strengths.

4.4 Bracing wall demand loads and ratios with wall strengths

Shelton (2007) states that NZS 3604 derives the demand load for Zone A using a base shear coefficient of 0.241. Thus, for the seismic weight of 58.3 kN, the room design demand base shear force was 58.3 x 0.241 = 14.05 kN.

Thurston and Beattie (2008a) recommended that the design strength of the veneer be taken as 0.7 times the calculated resistance, as given in Section 4.3 (= 0.7 x 21.4 = 14.98 kN). As noted above, the total resistance of the lining based on wall bracing tests was 10 kN for Stage I and 28 kN for Stage II. This data is plotted in Figure 13. The ratio of the NZS 3604 demand load to the design strength is therefore:

(a) Linings still fully effective – Stage I: Ratio = 14.05/(14.98 + 10) = 0.56

(b) Linings still fully effective – Stage II: Ratio = 14.05/(14.98 + 28) = 0.33

(c) Linings ineffective: Ratio = 14.05/(14.98 + 0) = 0.94.

It was concluded that the test room would need to be shaken with a significantly greater intensity than that corresponding to the NZS 4203 spectra to induce failure.

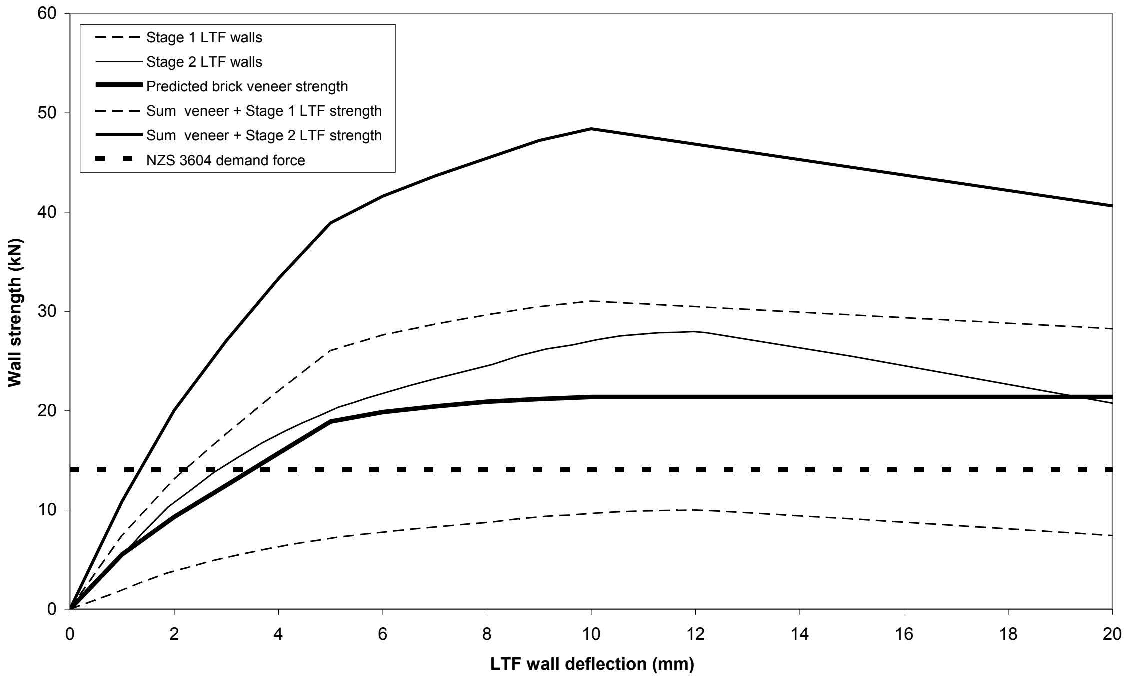


Figure 13. NZS 3604 demand load for Wellington and predicted strengths against deflection

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 **Error! Reference source not found.**. 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 servo valves and pump limited the maximum velocity to 250 mm/s. The peak applied simple harmonic acceleration was 0.77g.

Error! Reference source not found. also summarises the measured amplitude of ceiling horizontal deflection relative to the table. For instance in Test T23, 40 cycles of sinusoidal motion were imposed on the table at 4 Hz. The table displacement was ±4.7 mm and the peak ceiling deflections relative to the table were measured as ±13.2 mm. This used the Stage II lining fixing pattern (see Section 2) and Note 5 at the bottom of the table comments on this test. Tests not shown in the table imposed motion too small to be significant.

Figure 14 gives the number of cycles of ceiling deflection beyond discrete values of deflection. For instance, there were 120 cycles where the ceiling deflection was greater than 46 mm or more and 20 cycles where it was greater than 118 mm.

Table 1. Summary of shaking imposed and ceiling response

Test name	Stage *	Freq. Hz.	Table Movement ±(mm)	Calculated Velocity mm/sec	Calculated Accel. Ratio g	Number of cycles	Ceiling defl. relative to floor ±(mm)	Notes
T8	1	1	17.8	112.1	0.07	20	1.5	
T11	1	8	0.2	10.4	0.05	20	**	
T12	1	1	39.8	250.1	0.16	20	2.5	
T13	1	2	9.6	120.5	0.15	20	2.4	
T14	1	4	2.2	55.9	0.14	10	7.7	
T15	1	10	0.3	17.0	0.11	40	**	
T16	1	6	0.9	32.6	0.13	30	1.4	
T17	1	2	14.5	181.7	0.23	10	52.1	1
T17B	1	2	14.3	179.1	0.23	10	55.1	2
T18	1	4	3.0	76.2	0.20	20	4.6	
T19	1	8	0.7	36.4	0.19	40	**	
T20	1	2	18.6	234.1	0.30	20	46.8	
T21	1	1.5	25.9	243.9	0.23	20	82.6	
T22	2	2	17.2	216.0	0.28	20	4.3	3
T23	2	4	4.7	118.1	0.30	40	13.2	4
T24	2	4	7.7	193.5	0.50	40	17.3	
T25	2	2	19.0	238.8	0.31	20	58.7	5
T26	2	3	11.6	219.4	0.42	20	18.0	
T27	2	6	2.8	105.0	0.40	40	3.9	
T29	2	9	2.4	133.9	0.77	40	3.0	
T30	2	1.5	25.9	244.5	0.23	20	81.4	6
T31	2	1	39.8	250.3	0.16	20	118.0	7

Legend * See section 2 for a description of the differences between Stage 1 and Stage 2 construction
** Too small to measure

<u>**Notes:**</u>

1 Significant veneer cracks developed and panels rocked.
Side frame interior lining effectively detached from screws so there was negligable bracing provided.
Linings were rescrewed 50 mm away from original positions.

2 Linings detached again and same comments as 1 apply.
This is the same cyclic regime as Test T17.

3 Linings were replaced and a closer screw spacing used before this test commenced. The shaking was the same as T20 but the ceiling deflection was far less.

4 Some damage occurred to lining at screw connections.

5 This is the same cyclic regime as Test T22

6 Some ties near the veneer top developed small fatigue cracks.
This is the same cyclic regime as Test T21.

7 Many top ties broke or were damaged.
This is the same cyclic regime as Test T12 but the deflection is far greater as the lining has gone and the brick veneer was precracked.

5.2 Observations

5.2.1 General observations

Early shaking on Stage I construction was at low amplitude but slowly built up. Initially the veneer did not move, but at relatively low LTF wall deflections all panels rocked about a crack near the base. Crack patterns are described in Section 6. Early damage to the LTF plasterboard wall lining occurred during Test T14 (see **Error! Reference source not found.**). In subsequent shaking at 2 Hz the maximum LTF wall deflections increased from approximately 15 to 50 mm during the test as the linings detached from the framing. Subsequent cycles at 1.5 and 1.0 Hz resulted in maximum LTF wall deflections.

In all tests the LTF walls and veneer returned to near zero residual deflection when the shaking stopped, and the cracks in the veneer could simply be repaired by re-pointing apart from a few locations where bricks were dislodged due to the impacts of the rocking elements in Test T31.

When the LTF plasterboard lining connections failed the studs connected to the Panel F veneer bent significantly, but this was less noticeable elsewhere.

Based on scratch mark observations, the LTF frames neither lifted from the base connection nor slipped in shear.

Ties were noted to twist and distort during the tests. No ties slipped in the veneer mortar or failed in the screwed connection to the LTF framing. After the second to last test (i.e. Test T30), many ties near the top of the veneer were noted to have developed fatigue cracks partly across the tie at a location approximately 10 mm short of the tie-to-mortar interface.

In Test T31 Panel F initially rocked but then stopped rocking part-way through the test. This was attributed to the top three levels of ties (total of 15 ties) breaking at the fatigue crack noted above. At the end of this test one additional tie was noted to have broken, 16 ties had fatigue cracks partially through the ties and only three ties were uncracked.

Panels E and G had a total of 11 ties which had fatigue cracks partially through the ties. None were cracked right through and 24 were uncracked. Panels A and B had a total of 26 ties which had fatigue cracks partially through the ties, seven were cracked right through (these were adjacent to the window edge) and 12 were uncracked. No ties exhibited cracks in the end wall veneer panels.

5.2.2 Specific observations

The first test where any damage was detected or the ceiling deflection was significant was Test T14 where the ceiling deflection was ±7.7 mm relative to the table. The shaking was 10 cycles at 4 Hz. Figure 15 plots the deflection of the ceiling relative to the shake table and also the shake table movement for this test. The applied table movement consisted of two cycles of fade-in, 10 cycles to the target deflection and two cycles of fade-out. In the first five of the 10 cycles the ceiling relative deflection was slightly more than 180° out-of-phase with the floor deflection. The amplitude of ceiling motion then grew from approximately ±4 to ±7.5 mm because screws fixing the lining gradually weakened and therefore the natural frequency of the room increased closer to 4 Hz. The ceiling relative deflection gradually moved to 270° out-of-phase. Some very fine mortar cracks were observed during the shaking and, in one location on the veneer, small portions of mortar fell. Some damage was noted in the plasterboard fixing connections.

No additional damage was observed in the subsequent higher frequency tests of T15 and T16. However, in Test T17, where the excitation was at 2 Hz, the ceiling motion built up as the test progressed, as shown in Figure 16. Initially this was approximately ±15 mm which is close to the deflection corresponding to maximum wall strength. The amplitude of ceiling motion then grew to approximately ±51 mm as the wall lining became ineffective as bracing. Towards the end of the shaking the ceiling deflection decreased – probably due to room natural frequency changes. Additional screws were added to the side wall linings 50 mm from each original position. Test T17B repeated the same shaking. An almost identical result was achieved.

In both Test T17 and T17B wide veneer cracks and significant veneer rocking was observed. In the subsequent higher frequency tests, T18 and T19, the ceiling and veneer motions were small.

In Tests T20 and T21 very large ceiling and veneer rocking deflections occurred. The shaking frequencies were 2.0 and 1.5 Hz, respectively.

The linings were then replaced and a denser screw pattern was used to attach the lining to the LTF (see Figure 9(b)). This construction is called Stage II. The first test in Stage II (i.e. Test T22) used the same 2 Hz shake regime as Test T20 in Stage I, but the ceiling deflection was only ±4.3 mm (as against ±46.8 mm in T20) and no wall damage or brick rocking occurred. Clearly the stronger fixed lining resisted most of the load in Test T22, the veneer carried little load and the natural frequency has increased beyond 2 Hz.

Both Test T23 and T24 were at 4 Hz and the ceiling deflections were ±13.2 and ±17.3 mm, respectively. It is thought that the lining fixing had become weak after these tests. As there was a significant increase in shake displacement amplitude in Test T24, but only a small increase in ceiling deflection, it is thought that the room natural frequency had dropped below 4 Hz after Test T23 and thus resonance did not occur.

However, after some damage to the lining fixings was induced in Test T24 (where the excitation was at 4 Hz and ceiling relative deflections were ±17.3 mm), a repeat of the Test T22 input motion (in Test T25) resulted in ceiling relative deflections of ±58.7 mm. The ceiling motion built up as the test progressed (Figure 17), and the lining became completely ineffective as bracing.

Test T25 (which was at 2 Hz and the same shaking regime as Test T20 and T22) resulted in a large ceiling deflection (±58.7 mm), which is similar to that in Test T20 (±46.8 mm) and far greater than that in Test T22 (±4.3 mm).

Tests T26 (3 Hz), T27 (6 Hz) and T29 (9 Hz) induced relatively low ceiling displacements despite large velocities and accelerations. This showed that the response of the veneer, with the lining being ineffective, reduced with increased excitation frequency.

In Tests T30 and T31 the shaking frequency was 1.5 and 1.0 Hz respectively and the ceiling displacement was ±81.9 mm and ±113.2 mm respectively, showing that the room (with the linings now detached) responded most at low frequencies. Test T21 in Stage I and Test T30 had the same input motion, the lining was detached in both cases at the commencement of testing, and both had almost identical ceiling deflections of ±82 mm.

In both Stage I and Stage II, the failure of the lining occurred at the same excitation frequency of 2 Hz in Test T17 and Test T25. Although this same frequency was used in Test T13, the shaking amplitude was only half and no lining failure occurred.

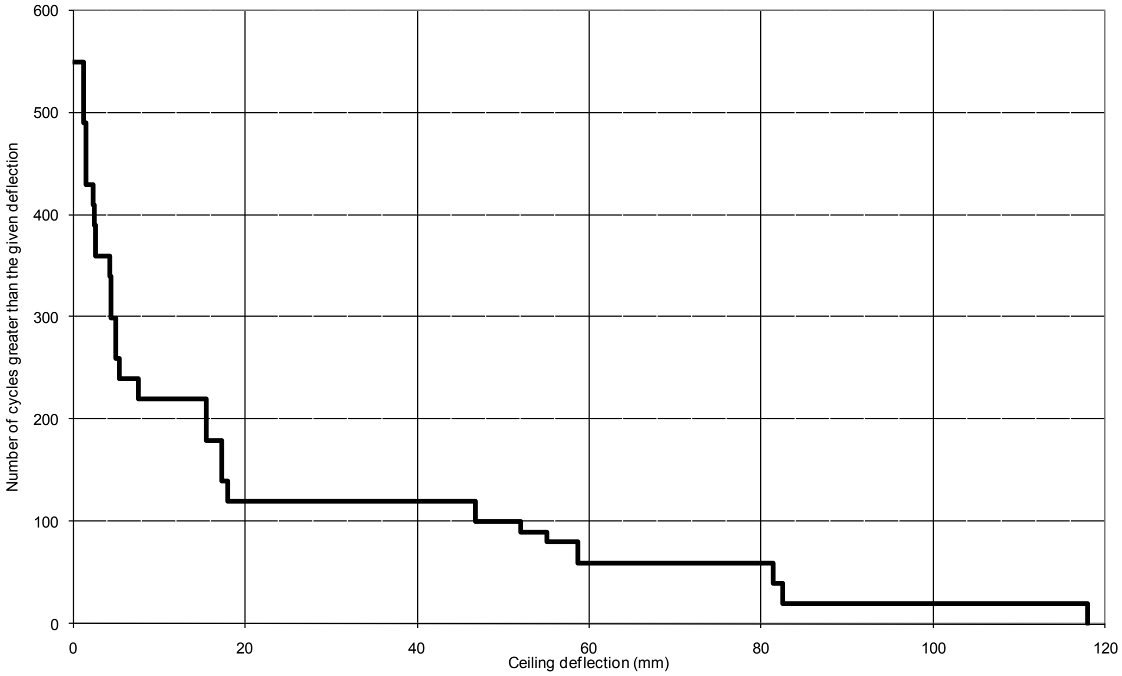


Figure 14. Number of excursions beyond discrete deflections

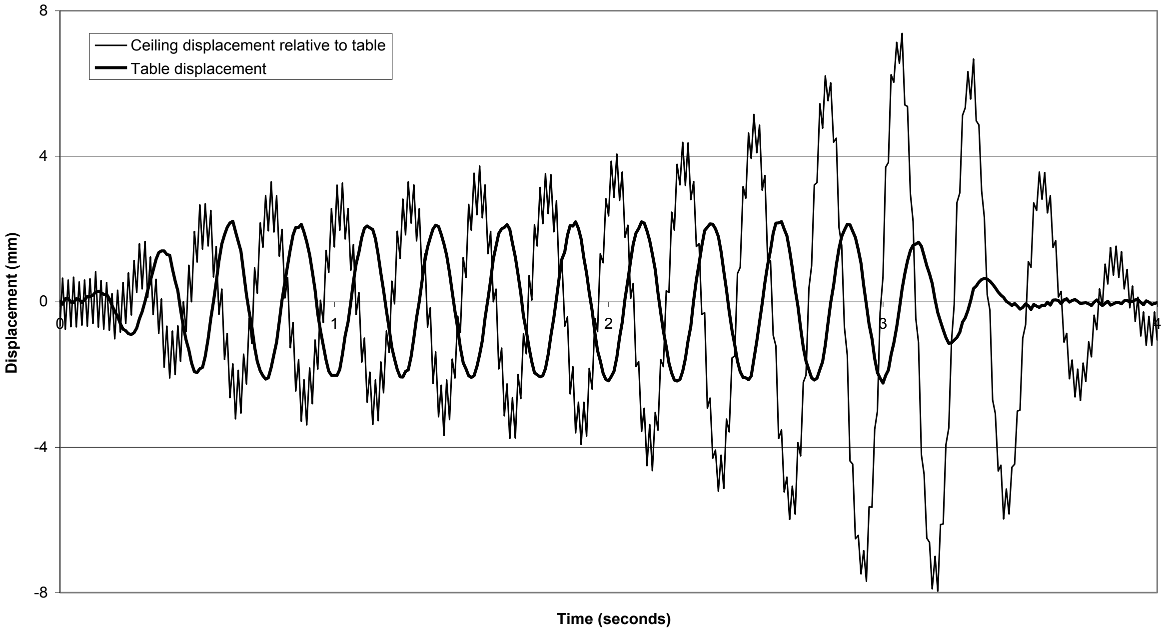


Figure 15. Table and ceiling displacements in Test T14 (cycling frequency 4 Hz)

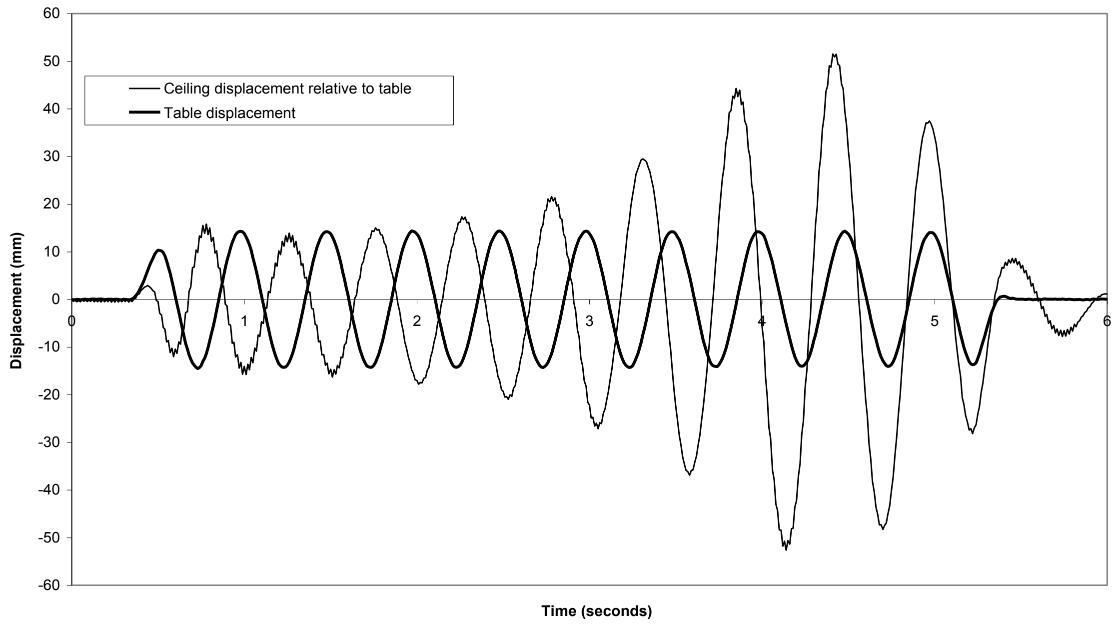


Figure 16. Table and ceiling displacements in Test T17 (cycling frequency 2 Hz)

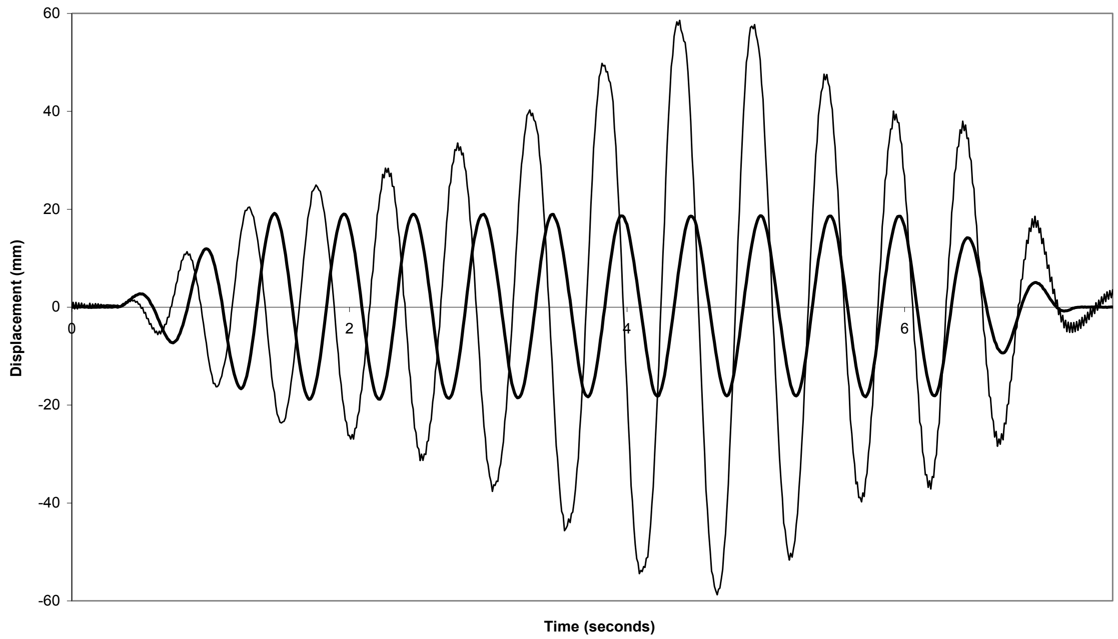


Figure 17. Table and ceiling displacements in Test T25 (cycling frequency 2 Hz)

6. VENEER CRACKING

A three dimensional view of the observed veneer cracking at the end of Test T17 is shown in Figure 18(a). This is the first test where significant veneer cracking occurred. The amplitude of ceiling deflection reached ±52 mm. Figure 18(b) shows the cracking at the completion of the entire test room shaking program.

All four “L-shaped corner veneer elements” and the single isolated “pier” (Panel F) rocked about alternate ends of the cracks shown during the backward and forward motion. A few bricks cracked and fell out (see Figure 18(b) for location). Increased rocking generally did not alter the crack pattern, except for two major cracks which occurred during the last test (Test T31) which are shown in Figure 18(b).

Figure 19 and Figure 20 show Side 1 of the veneer at peak forward and backward deflections in Test T31. Figure 21 and Figure 22 are corresponding views of Side 2. The rocking motions can be readily seen.

In Test T31, Panel F initially rocked but then this action stopped as the ties fatigued near the top of this panel because of the large number of large displacement cycles that had been applied, and the panel stopped following the ceiling deflection.

Figure 23 to Figure 31 show the condition of the veneer at the end of all shake testing. All panels finished close to their initial uncracked positions. Although some of the cracks were unsightly they could all be re-pointed. Many ties had failed or had fatigue cracks, but this was because of the very large number on large deflection cycles imposed (as was noted for Panel F above).

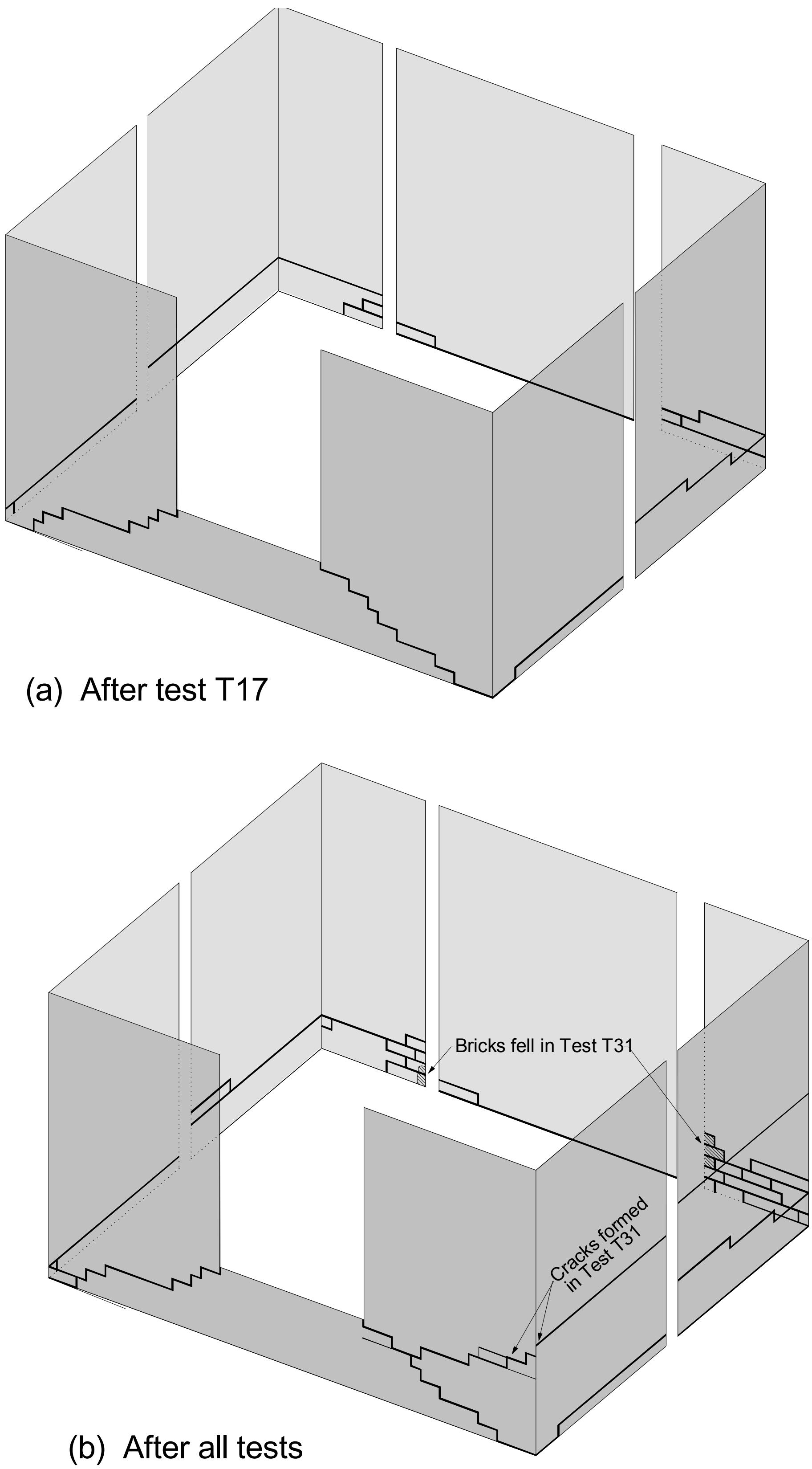


Figure 18. Three dimensional view of veneer crack patterns

Figure 19. Cracks on Side 1 at peak forward deflection in Test 31

Figure 20. Cracks on Side 1 at peak backward deflection in Test 31

Figure 21. Cracks on Side 2 at peak forward deflection in Test 31

Figure 22. Cracks on Side 2 at peak backward deflection in Test 31

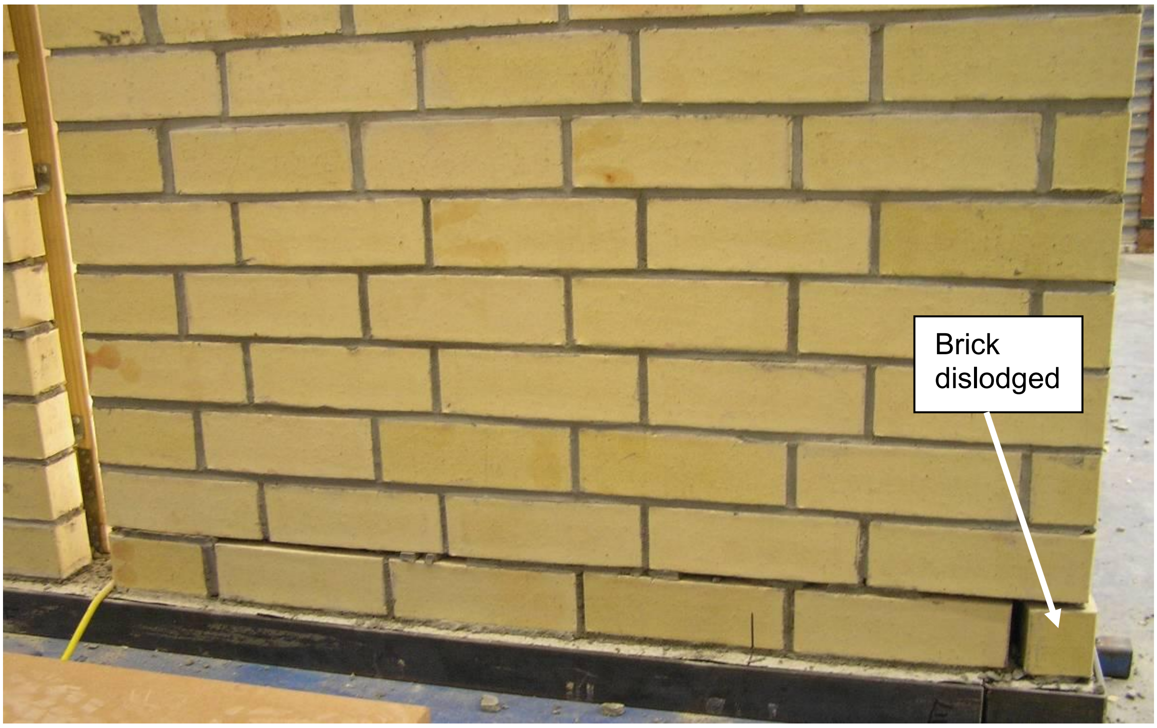


Figure 23. Typical condition of brick veneer at room ends at test completion (face-loaded panels)

Figure 24. Condition of Panel G at test completion (in-plane-loaded panel)

Figure 25. Condition of Panel F at test completion (loaded in-plane)

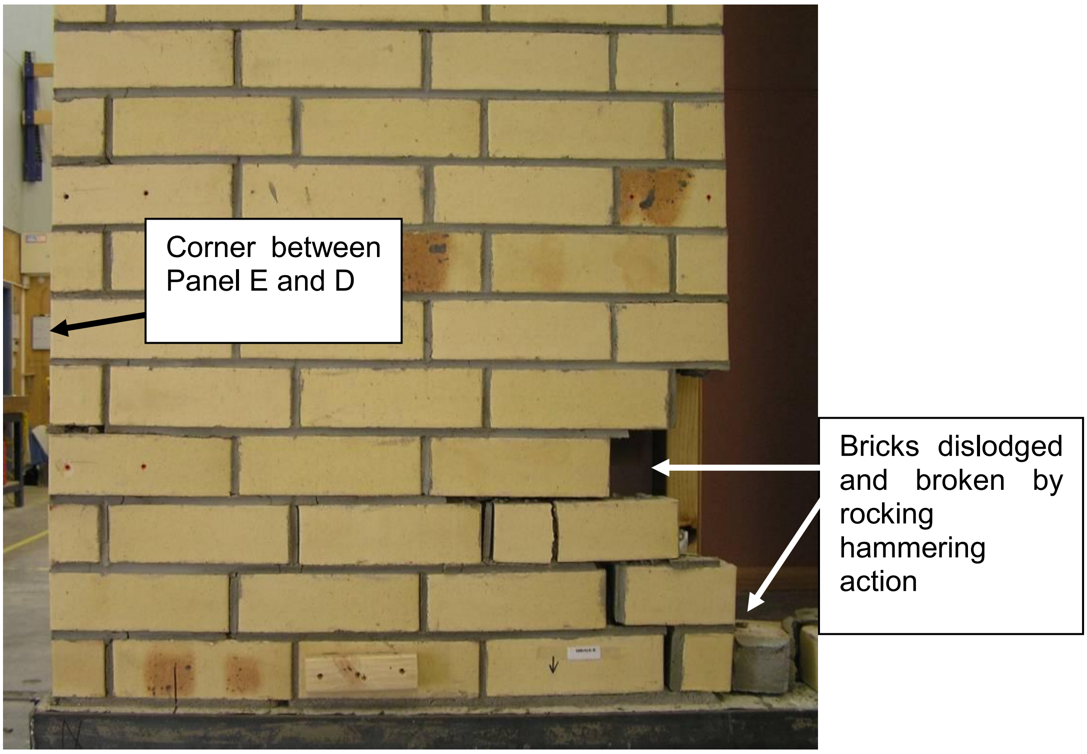


Figure 26. Condition of Panel E at test completion (loaded in-plane)

Figure 27. Condition of Side 2 and End 1 at test completion

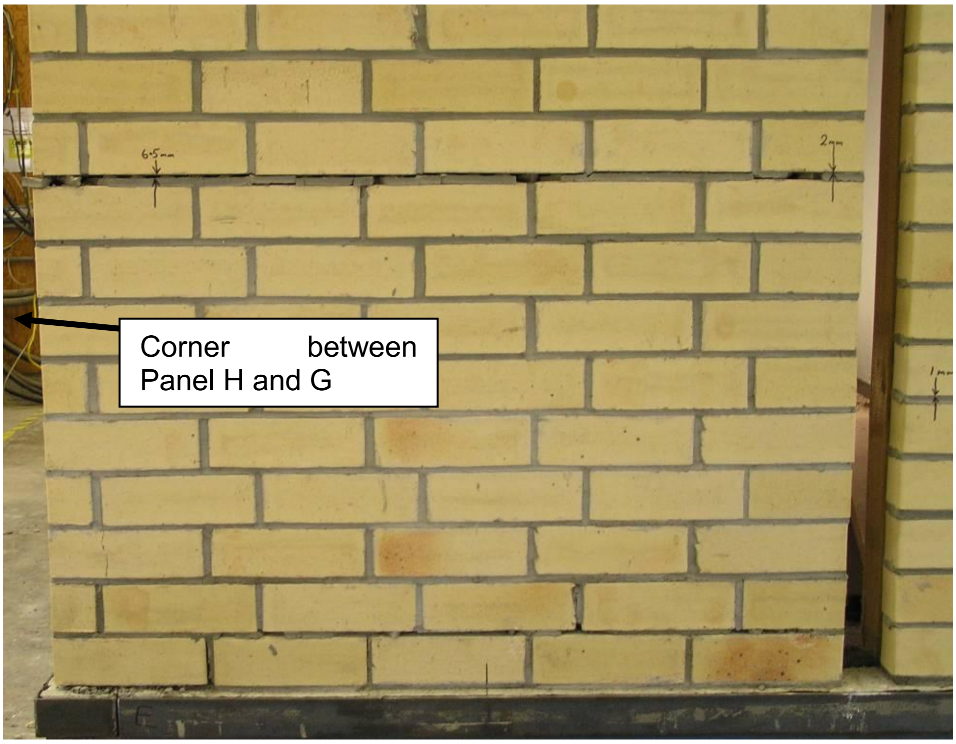


Figure 28. Condition of Panel H End 1 at test completion (face-loaded)

Figure 29. Condition of Side 1 (Panels A and B) at test completion (loaded in-plane)

Figure 30. Condition of Panel B at test completion (loaded in-plane)

Figure 31. Condition of Panel A at test completion (loaded in-plane)

7. MEASURED DISPLACEMENTS 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 32 plots the amplification ratio (= measured ceiling displacement relative to table divided by the table displacement amplitude) versus table displacement frequency for tests where the lining was still well attached and where the ceiling displacement was sufficient that it could be accurately measured. Peak values in Figure 32 are at 4 Hz, whereas the natural frequency found in Section 9 from the measurements of decay vibrations was 7.5 Hz. However, the peak in Figure 32 is not well defined as insufficient runs were performed to create an accurate curve. There are three peak points plotted at 4 Hz with the two lower values being from Tests T23 and T24, where the lining fixing had softened and ceiling displacements had reached 13.2 and 17.3 mm. It is concluded that the natural frequency of the room with lining is sensitive to the preceding loading.

As most of the analysis in this report using the natural frequency for the case of the lining in place uses data from tests where the ceiling displacement amplitude varied between 7–17 mm, the natural frequency for this state is taken as 4–5 Hz.

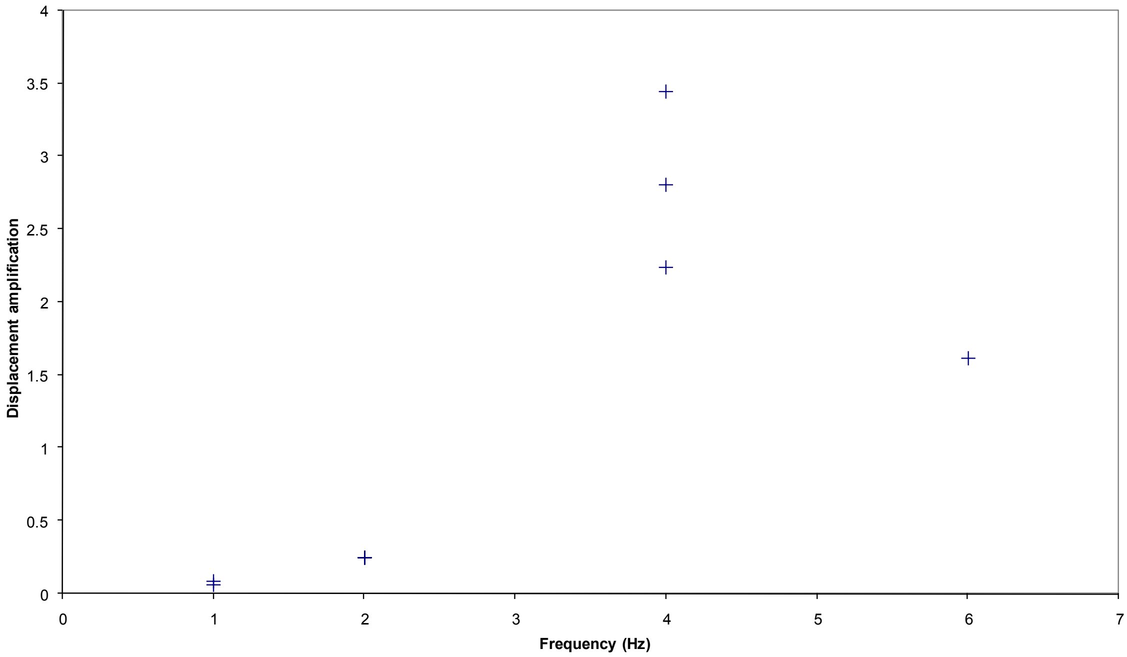


Figure 32. Natural frequency of room when linings were firmly fixed

Figure 33 plots the amplification ratio for tests where the wall lining was effectively detached from the framing. The resistance is then due only to the brick veneer. As there were many lengths of veneer, and as veneer panels have a non-linear force/displacement relationship, a simple natural frequency is not expected. From Figure 33 the best estimate of natural frequency is approximately 1.5 Hz. Three ratios are plotted at 2 Hz. The highest ratio occurs in Test T17 where the linings first came loose in Stage I and the linings may still have had some influence and increased the natural frequency. The next highest ratio occurs in Test T25 where the linings first came loose in Stage II and the same conclusion applies. Thus, the lowest point is expected to be the most reliable indication for the detached lining situation and this fits the curve of the other data best.

It is concluded that the natural frequency of the room was approximately 4–5 Hz when the lining was well attached and was experiencing displacement amplitudes in the order of 7–17 mm.

It is further concluded that the room natural frequency was between 1.25 and 1.75 Hz when the lining was effectively detached. However, unlike an elastic system, it did not display a sharp increase in displacements at the natural frequency, but rather shaking in the 1 to 2 Hz range gave similar displacement amplifications.

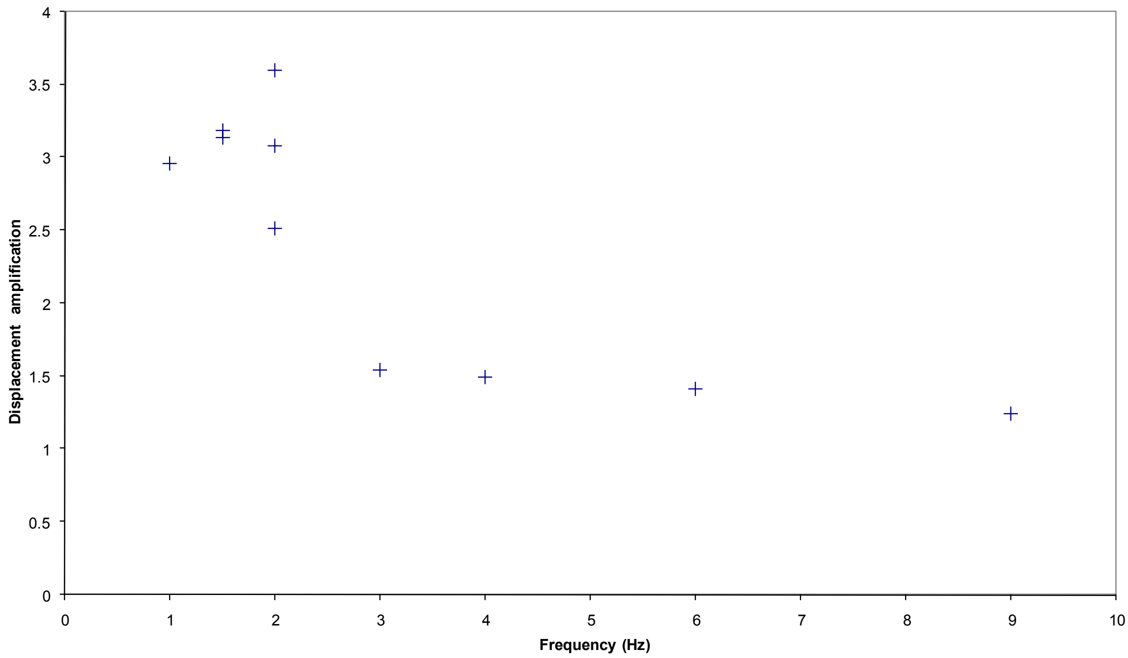


Figure 33. Natural frequency of room when linings effectively not fixed

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 deformation 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 ties did, however, rotate on the screw connection to the LTF. The in-plane differential displacement (i.e. X where the movement is $\pm$X mm) is plotted in Figure 34. It can be seen that the tie deformation was small for each of the "L-shaped corner veneer elements". This corresponded to the observations and subsequent theory by Thurston and Beattie (2008a) where it was considered that the ties connecting the end face-loaded veneer 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. However, large tie deformation occurred at the isolated veneer panel (Panel F).

The horizontal deflection, Δ, of a brick veneer panel induced by rocking is given by:

$$\Delta = (V_A - V_B) \times H / L_{AB} \quad \text{.........} \quad (7\text{-}1)$$

Where

H	=	the panel height about the pivot point
V_A, V_B	=	the vertical displacements measured at the two panel ends, A and B
L_{AB}	=	the distance between points A and B.

Figure 35 plots the amplitude of panel rocking displacement versus the applied table displacement for all monitored in-plane-loaded panels. The rocking displacement of the "L-shaped corner veneer elements" was approximately 2.25 times the table displacement. However, the isolated veneer Panel F only rocked to an amplitude approximately 1.25 times the table displacement.

The amplitude of panel rocking displacement is plotted in Figure 36 versus the measured ceiling displacement. The rocking displacement of the “L-shaped corner veneer elements” was approximately 0.8 times the ceiling displacement relative to the table.

7.3 Base slip

Figure 37 plots the amplitude of panel horizontal base slip versus the applied table displacement for all in-plane-loaded panels. The base slip of the “L-shaped corner veneer elements” was almost zero. This corresponded to the observations and subsequent theory by Thurston and Beattie (2008a). However, the isolated veneer Panel F exhibited significant base slip.

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

The horizontal out-of-plane differential displacement 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. The peak veneer out-of-plane movement is plotted in Figure 38 with negative values being the veneer moving 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 34 (as would be expected from the theory by Thurston and Beattie (2008a)).

The range of movement in each test is plotted in Figure 39. For example, in Test T31 the top of Panel C End 2 was closer to the LTF by between 8.19 and 13.27 mm during the test. Thus, a point at -13.27 mm is plotted in Figure 38 at the ceiling displacement of 118 mm and a point at (13.27-8.19 =) 5.04 mm is plotted in Figure 39 at the ceiling displacement of 118 mm. The measured gap between the walls did not increase by more than 1 mm in any of the tests. A comparison of the plots in Figure 38 and Figure 39 shows that the ties never fully straightened after buckling.

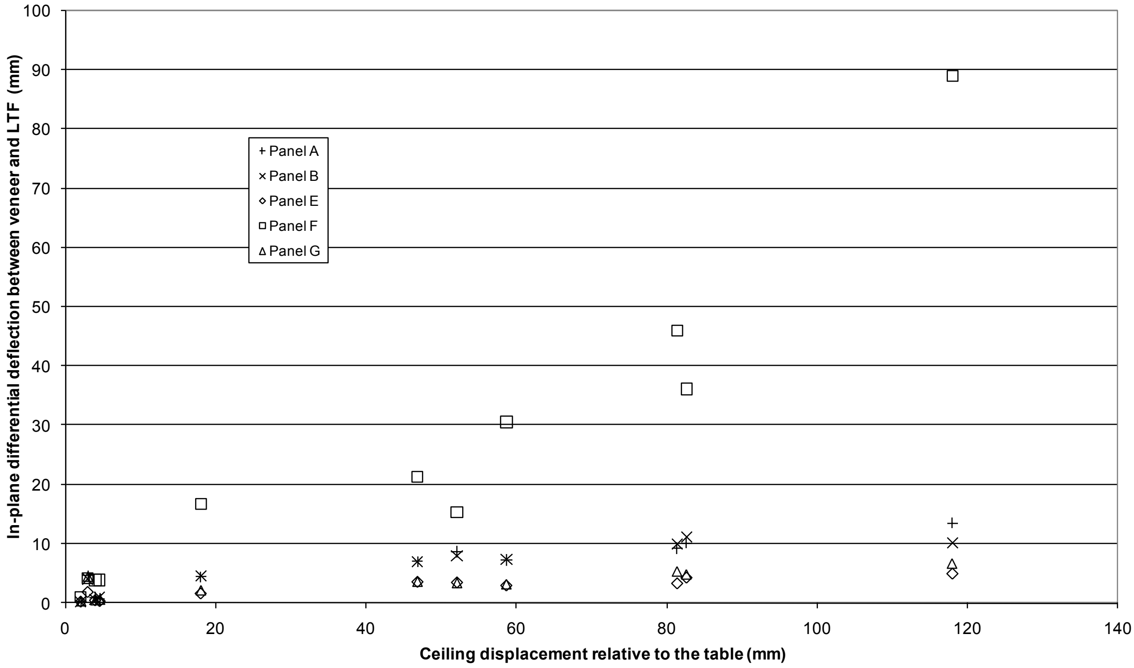


Figure 34. Tie deflection versus ceiling displacement

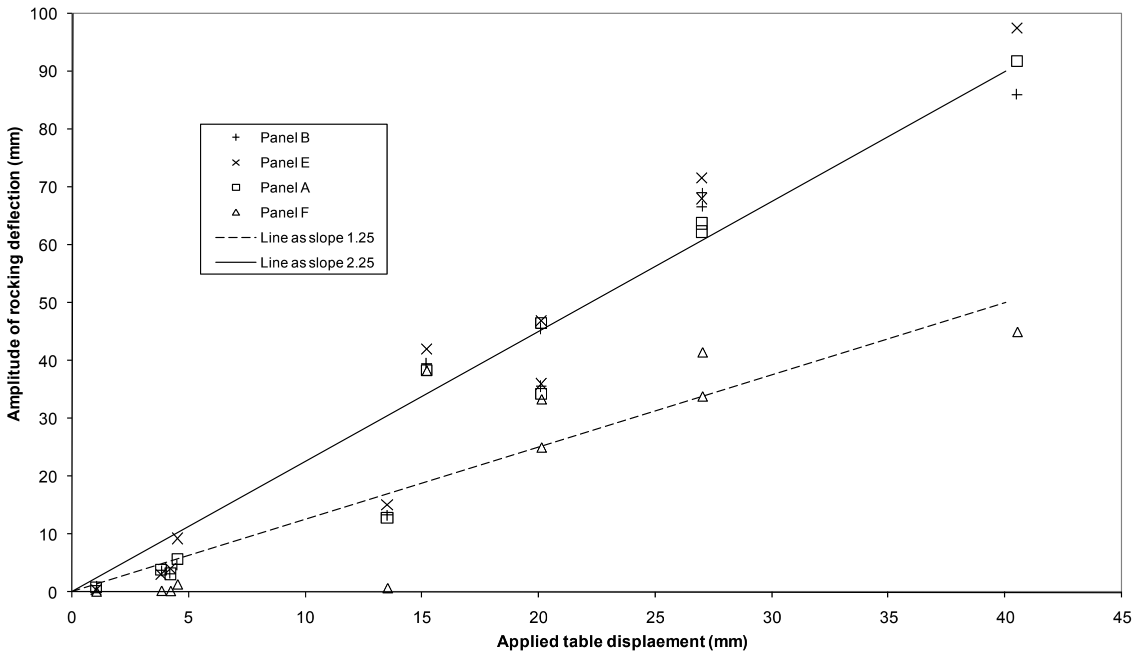


Figure 35. Panel rocking deflection versus applied table displacement

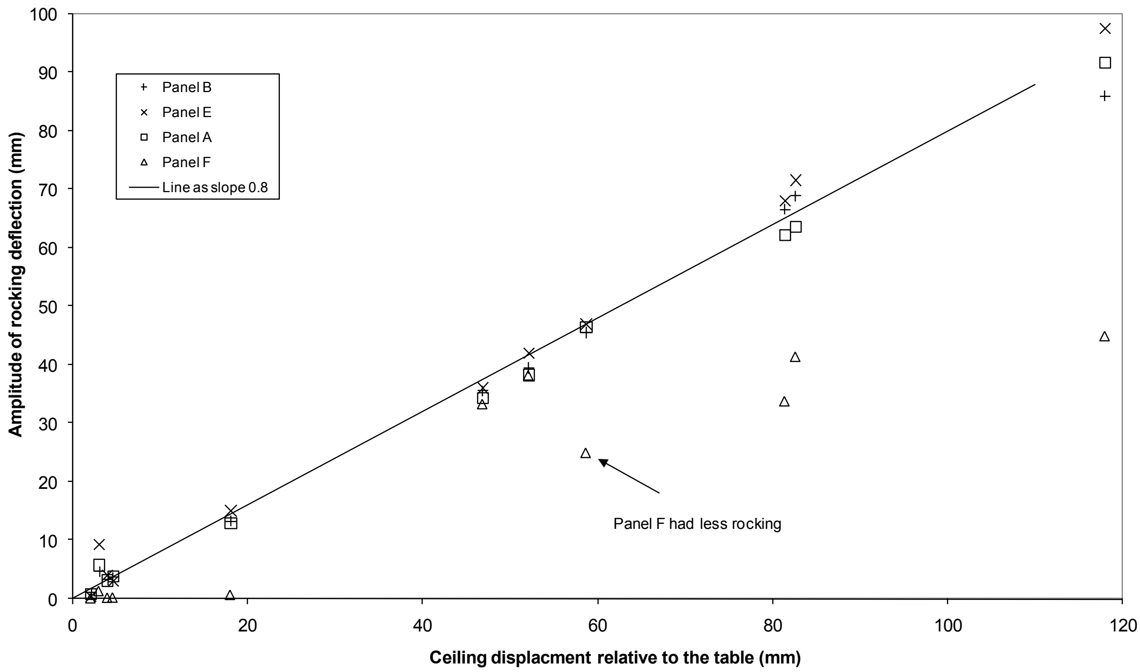


Figure 36. Panel rocking deflection versus ceiling displacement

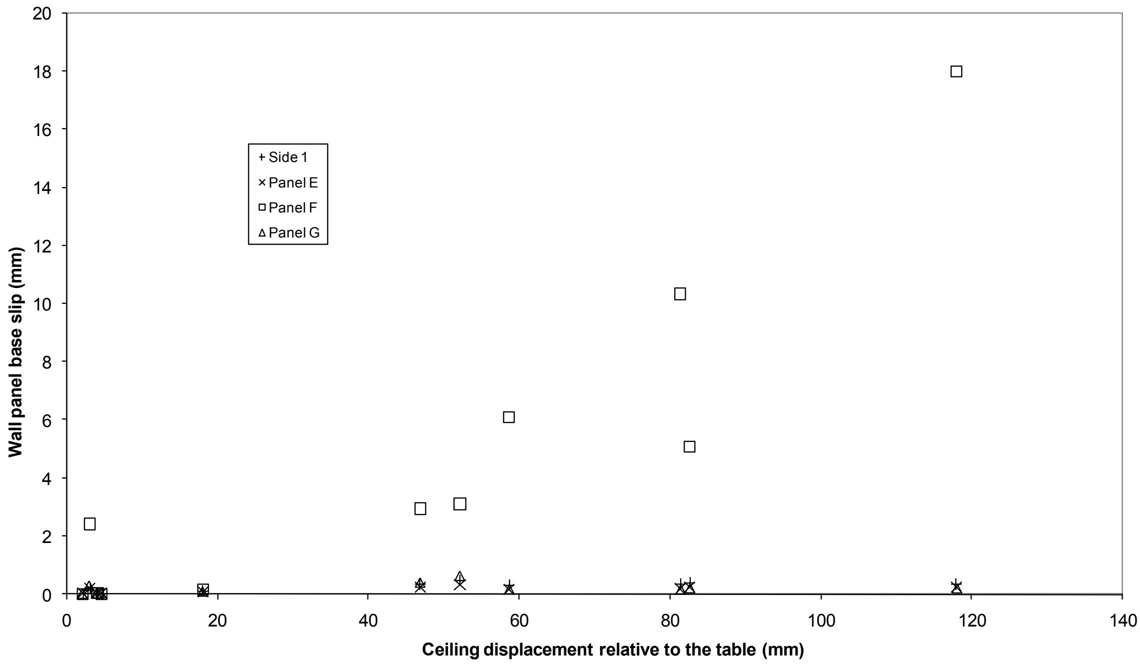


Figure 37. Panel base slip versus ceiling displacement

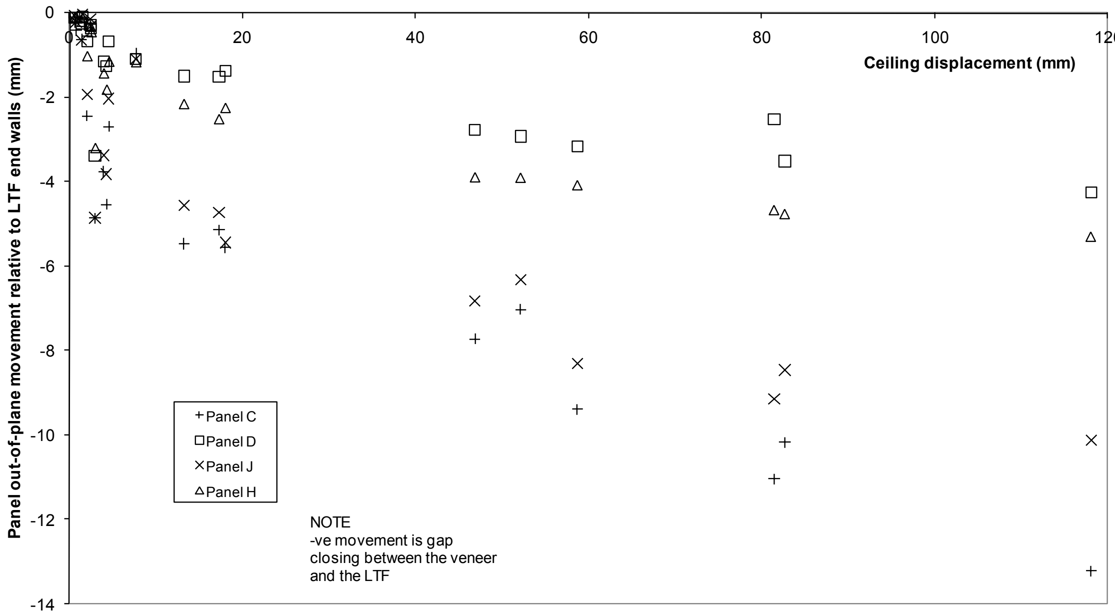


Figure 38. Maximum veneer panel out-of-plane displacement relative to LTF

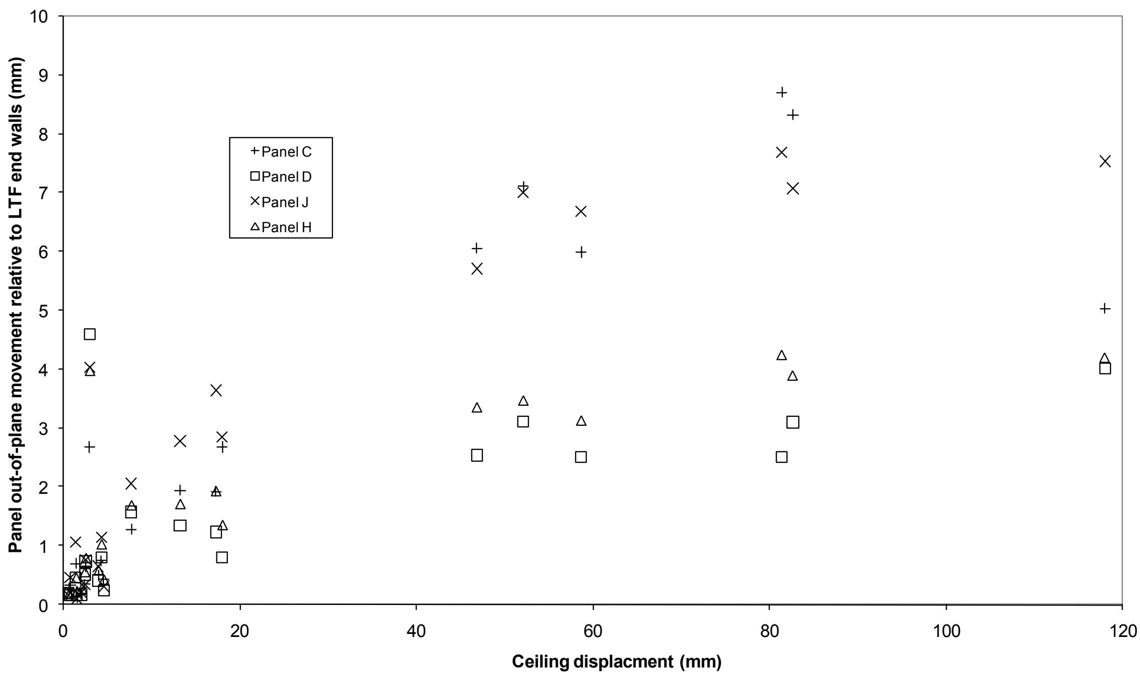


Figure 39. Range of veneer panel out-of-plane displacement relative to LTF in each test

8. MEASURED ACCELERATIONS IN SHAKE TABLE TESTS

Typical plots of measured table, ceiling and top of veneer acceleration time histories are shown in Figure 40 to Figure 42, respectively. It can be seen that the table accelerations are far from sinusoidal and the ceiling accelerations are smoother and closer to sinusoidal.

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 table accelerations experienced. As the records of accelerations exhibited spikes, particularly when the rocking veneer panels impacted the table, the dynamic amplification factor was taken as the ratio of the RMS (root mean square) of the measured accelerations.

Figure 43 plots the ratio of ceiling RMS acceleration to table RMS acceleration versus ceiling displacement. 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 approximately 1.0 for 10 mm ceiling deflection and approximately 2.0 for 120 mm ceiling deflection.

Figure 44 plots the ratio of the RMS acceleration at the top of the veneer to table RMS acceleration versus ceiling displacement. Again it can be seen that there is a considerable scatter of results. For ceiling displacements greater than 10 mm, a best fit straight line gave a ratio of approximately 1.6 for 10 mm ceiling deflection and approximately 2.4 for 120 mm ceiling deflection.

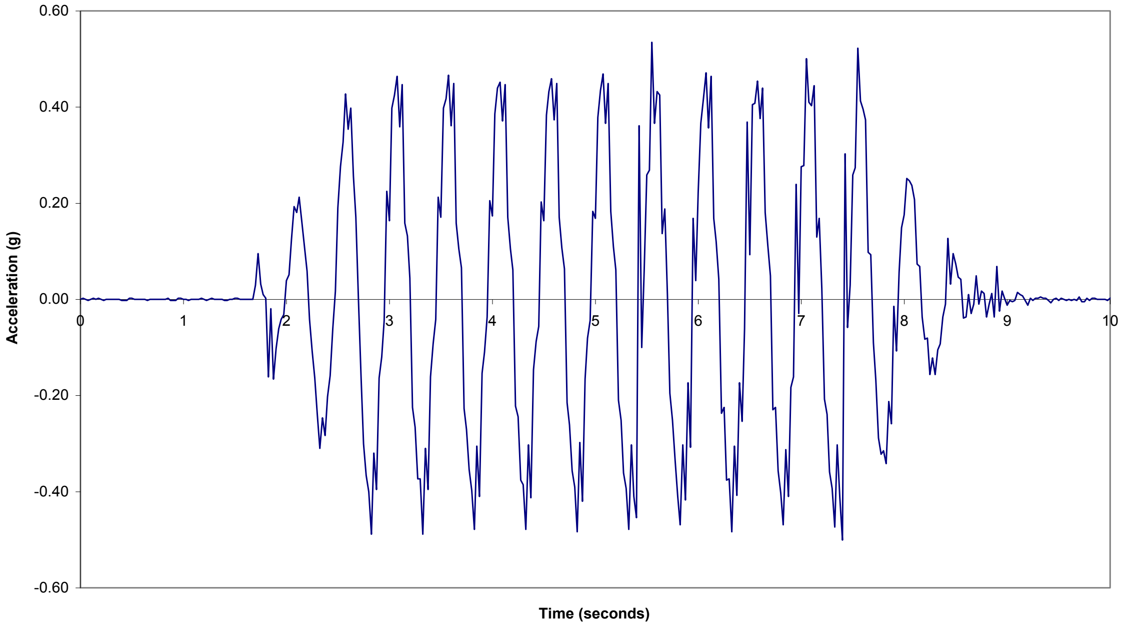


Figure 40. Measured table acceleration in Test 17

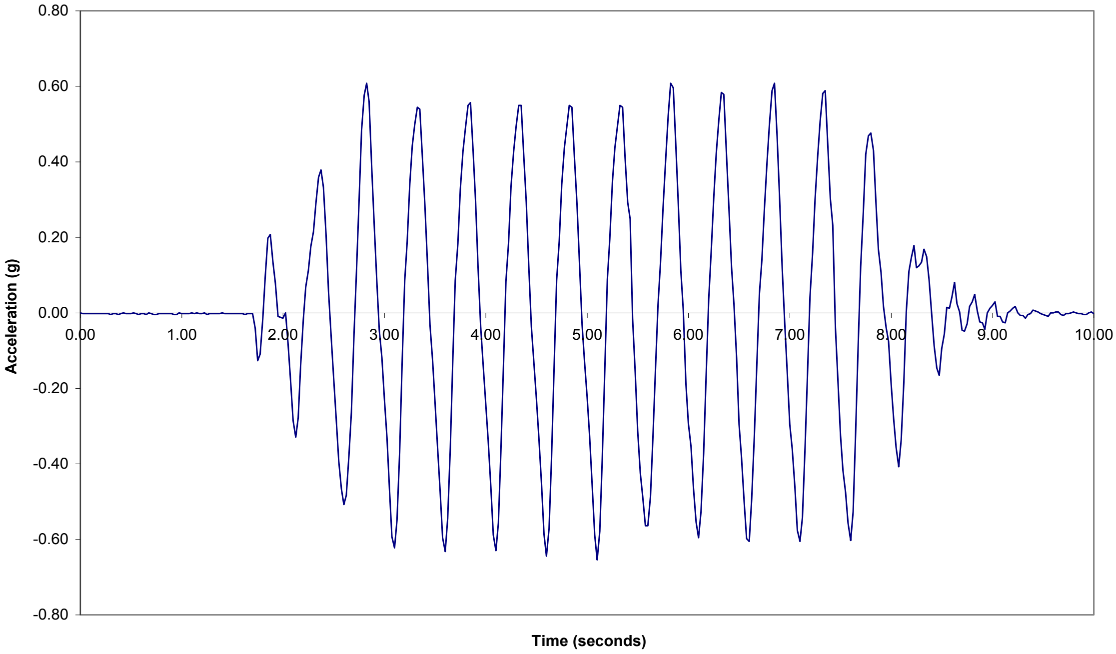


Figure 41. Measured ceiling acceleration in Test 17

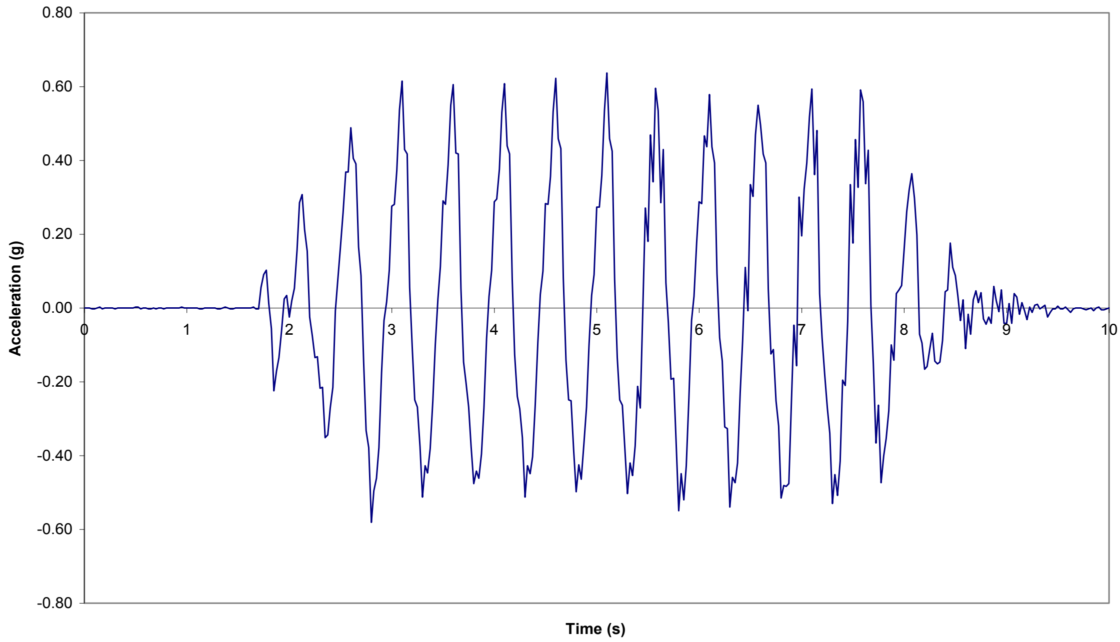


Figure 42. Measured acceleration at the top of Side 1 veneer in Test 17

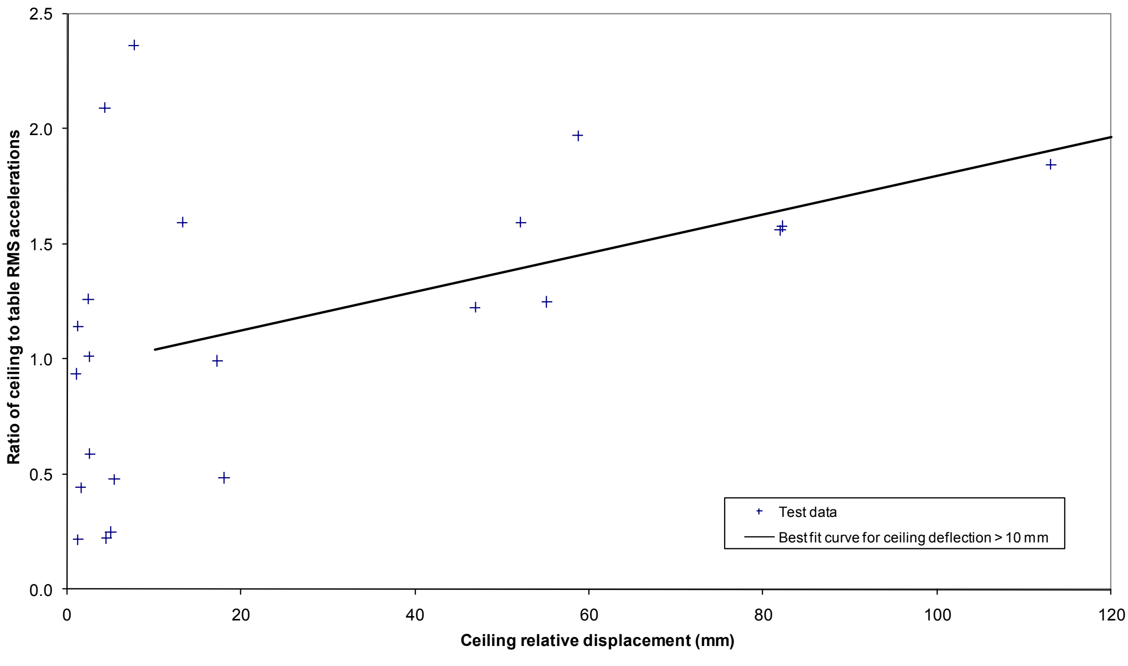


Figure 43. Ratio of ceiling RMS acceleration to table RMS acceleration versus ceiling displacement

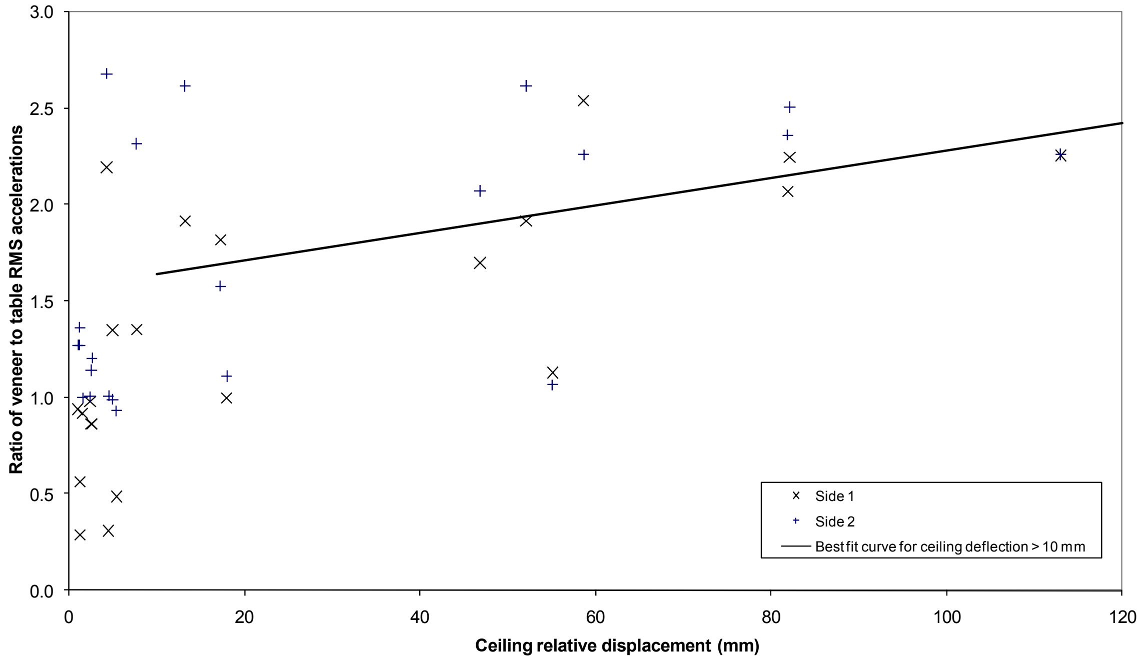


Figure 44. Ratio of veneer RMS acceleration to table RMS acceleration versus ceiling displacement

9. NATURAL FREQUENCY AND DAMPING

The natural frequency and damping of the test room was determined from the decay vibrations by either impacting the frame at the ceiling level with a rubber mallet and recording the displacement response or from a record of the residual motion after the table had stopped moving. The analysis used the well-known formulae (Chopra undated). Results are shown in Table 2.

Table 2. Measured room natural frequency and damping

	Frequency (Hz)	Damping
Stage I Lining		
Before testing	7.5	3.7%
After Test T15	5.5	6.7%
After Test T17	3.6	6.4%
Stage II Lining		
After Test T21	7.5	4.5%
End of all tests	1.8	7.0%

Initially the natural frequency of the room was 7.5 Hz, but this decreased to 3.6 Hz after Test T17 when the linings became ineffective as bracing elements. The damping increased to 6.4%.

The walls were then relined for the Stage II tests and the natural frequency reverted back to 7.5 Hz. The damping was measured at 4.5%. However, at the end of all testing, when the lining had completely detached from the framing, the natural frequency of the room had dropped to 1.8 Hz and the damping increased to 7.0%.

The measured values of natural frequency and damping were for small displacement offsets with no veneer rocking. When the veneer rocked it was shown in Section 7.1 that the natural frequency was probably 1.25 to 1.75 Hz. The damping for this type of action is mainly due to hysteresis. Non-linear computer models usually assume 5% damping. In Figure 45 to Figure 50, the 10% and 20% damping spectra have also been plotted to show the sensitivity of the response to damping.

10. 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 T22) is shown in Figure 45. The excitation frequency in this test was 2 Hz (Period = 0.5s). The ordinate at the peak for the 5% damping curve is 3.29g. Thus, a single degree of freedom (SDOF) elastic oscillator, carrying a weight of 10 kN, with natural period of precisely 0.5s and damping of 5% will have a peak base shear force of 10 x 3.29 = 32.9 kN when subjected to the measured table accelerations.

Figure 46 shows the total response spectra obtained by accumulating all the shaking up to the end of Test T16 (i.e. before any significant veneer cracking or lining fixing deterioration had occurred). Also plotted in Figure 46 are the NZS 4203 elastic design spectra for Wellington factored by 1, 2 and 3. These have been calculated by multiplying the basic seismic hazard acceleration coefficient for an intermediate soil site by the Zone factor for Wellington (= 1.2). Other terms in the equation for lateral force coefficient determination have been taken as unity. In Section 7.1, it was shown that the room natural period for this level of shaking, when the wall linings were still effective, was between 4 and 5 Hz (i.e. 0.25 to 0.20s period). Considering this zone of Figure 46, it can be seen that the shaking imposed corresponds to between 1.0 and 2.0 times the design spectra.

The lining became ineffective in Test T17. The 2.0 Hz shaking in Test T20 induced relative ceiling displacements of 46.8 mm and the 1.5 Hz shaking in Test T21 induced ceiling displacements of 82.6 mm. The sum of the response spectra after these tests is given in Figure 47. In Section 7.1 it was shown that the room natural frequency for this level of shaking, when the wall linings were still effective, was between 1.25 and 1.75 Hz (i.e. a period of 0.8 to 0.57s). Furthermore, the response was not particularly sensitive to the shaking frequency in this range. Thus, considering the zone in the frequency range between 1.25 and 1.75 Hz of Figure 47, the shaking for Stage I is considered to be at least 3.5 times the NZS 4203 design spectra.

The lining was replaced in the Stage II tests. Figure 48 shows the total response spectra from all the shaking in Stage II up to the end of Test T24 (i.e. before major wall lining fixing deterioration had occurred). This was clearly quite severe at the expected natural frequency of 4 Hz, but the natural frequency may have reduced below this as there was some wall lining damage and the wall racking deflection in Test T24 reached ±17.3 mm. The shaking survived by the room is estimated to be between 1.0 and 8.0 times the design spectra.

Figure 49 shows the total response spectra from all the shaking up to the end of Test T30. At this stage some ties had developed small fatigue cracks but none had broken. In the frequency range between 1.25 to 1.75 Hz of Figure 47 the shaking is considered to be at least 4.0 times the NZS 4203 design spectra. Note: the lowest value in this region is not taken as Section 7.1 showed that the response was not very sensitive to shaking frequency.

Figure 50 shows the total response spectra from all the shaking in the complete test series. It is very similar to Figure 49 except the ordinate at 1.0s is slightly greater. The shaking is considered to be equivalent to at least 4.0 times the NZS 4203 design spectra.

In summary, in the shaking of Stage I on the room survived without significant lining damage between 1.0 and 2.0 times the NZS 4203 ultimate limit state (ULS) design shaking for houses. In Stage II it was between 1.0 and 8.0 times the ULS shaking, which is too large a range to be useful.

After the linings had become ineffective (i.e. detached from the framing) the shaking survived by the veneer was estimated to be at least 3.5 times the NZS 4203 design spectra in Stage I and 4.0 times in Stage II.

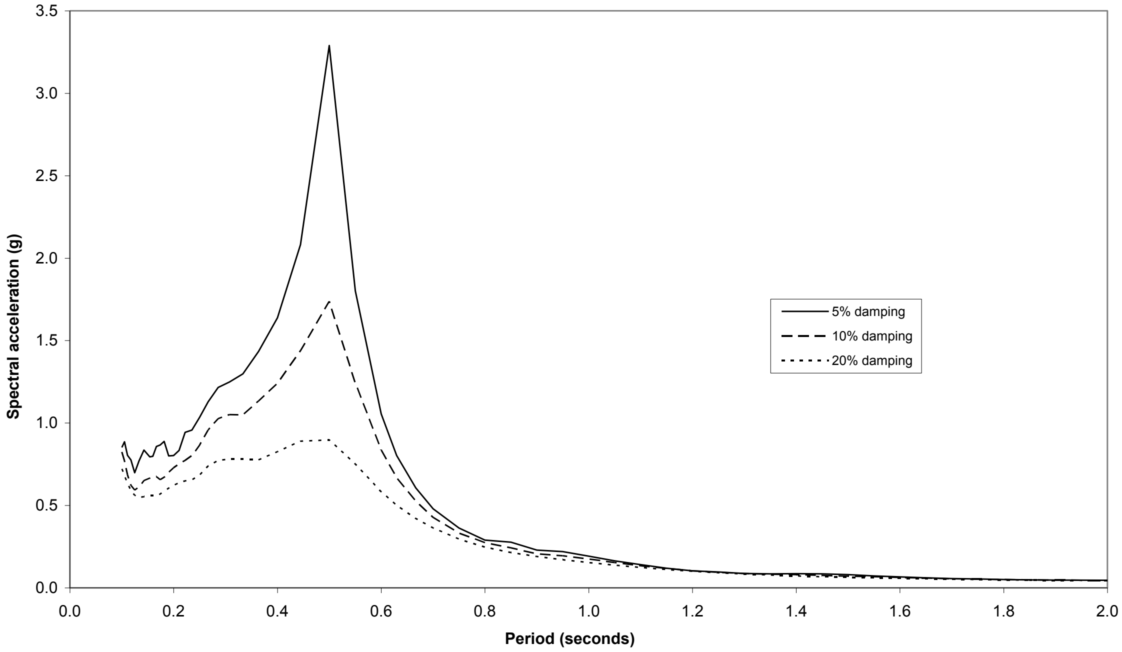


Figure 45. Response spectra from shaking of Test T22

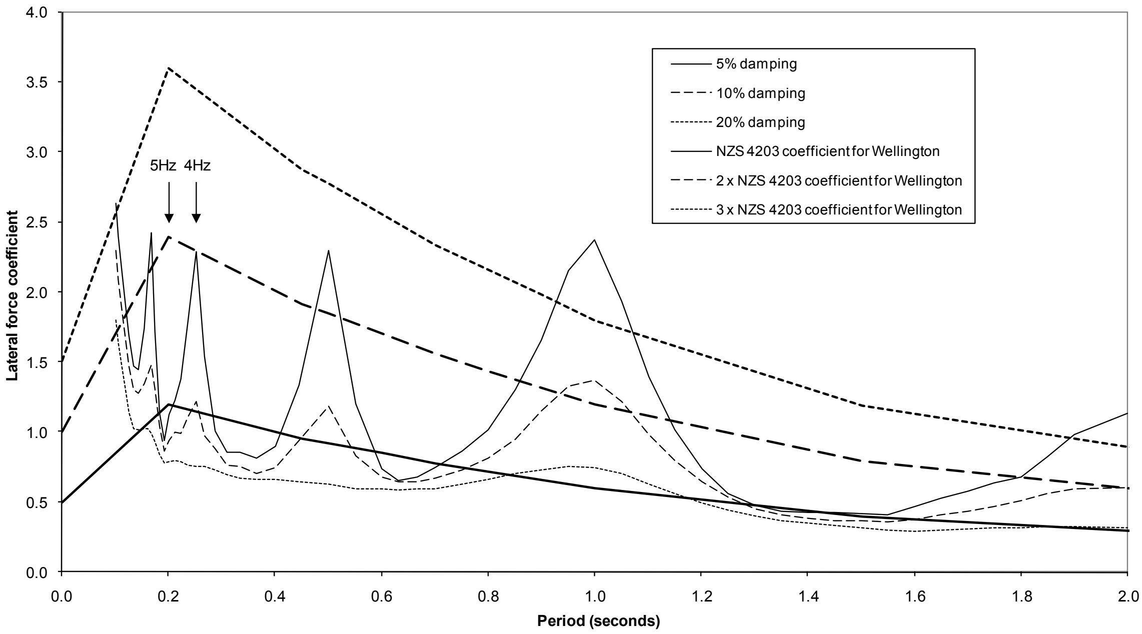


Figure 46. Summation of response spectra before significant veneer cracking in Stage I

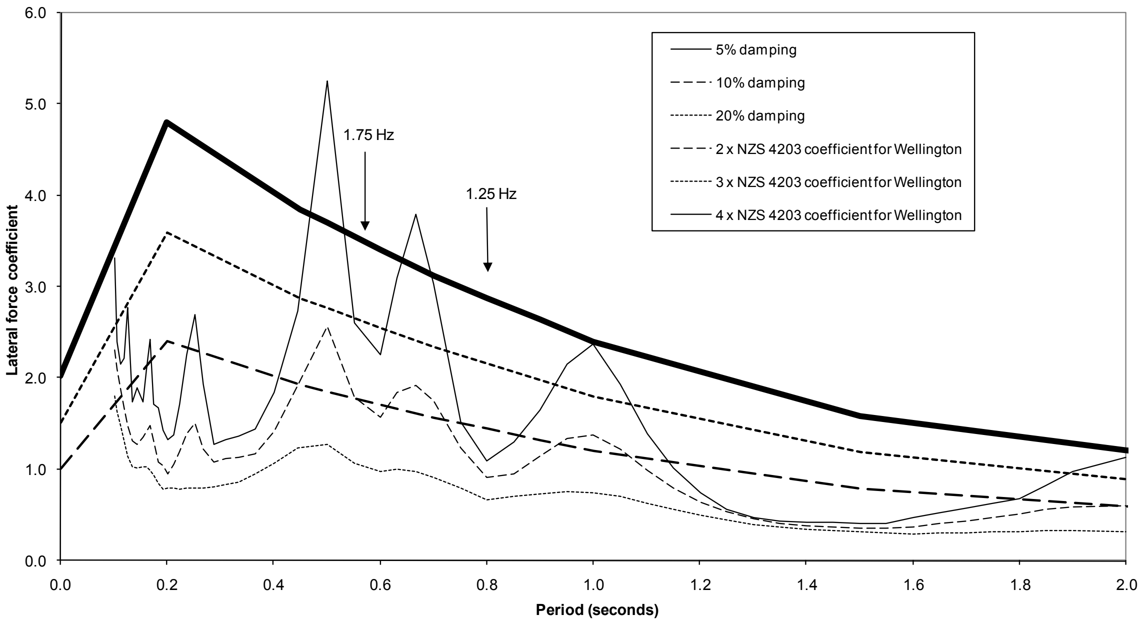


Figure 47. Summation of response spectra in Stage I

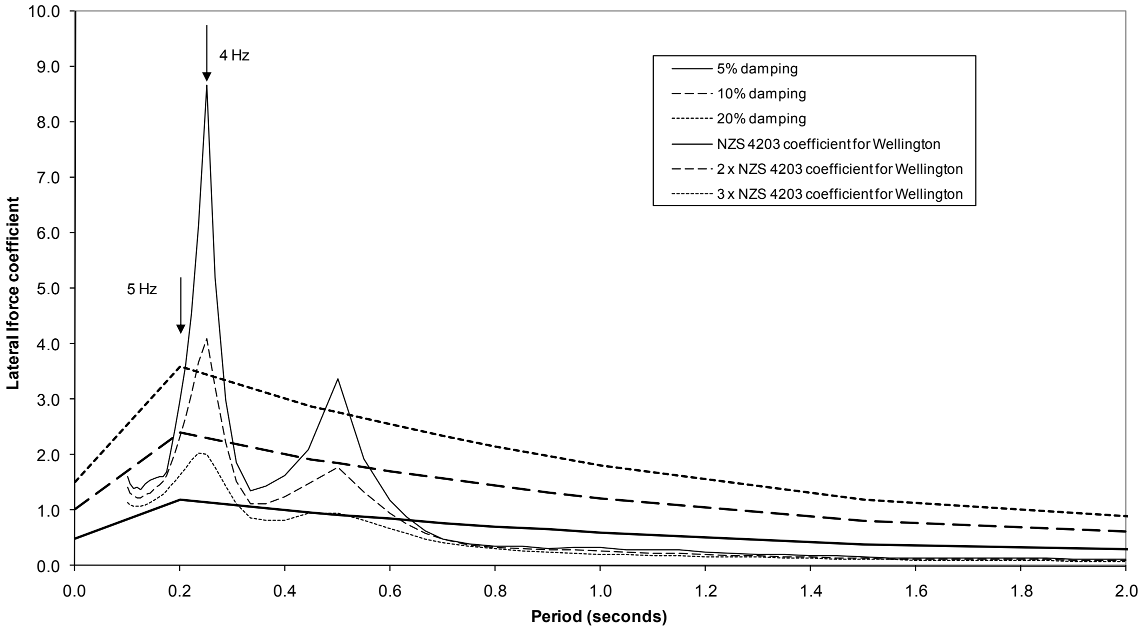


Figure 48. Summation of response spectra in Stage II before significant veneer cracking

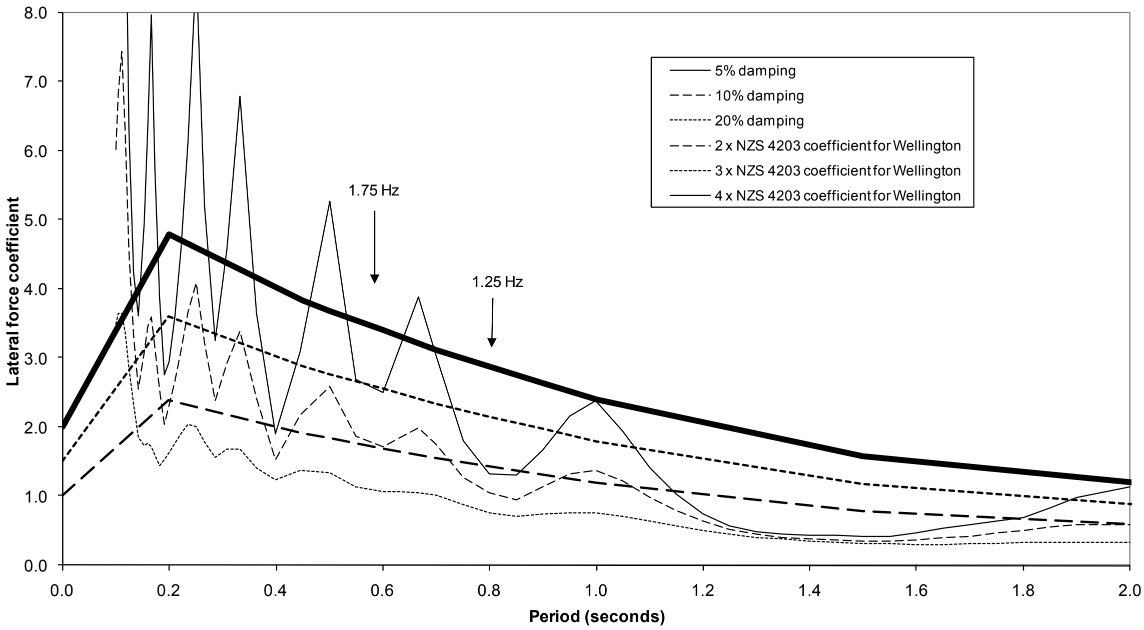


Figure 49. Summation of response spectra in Stages I and II before ties fatigued

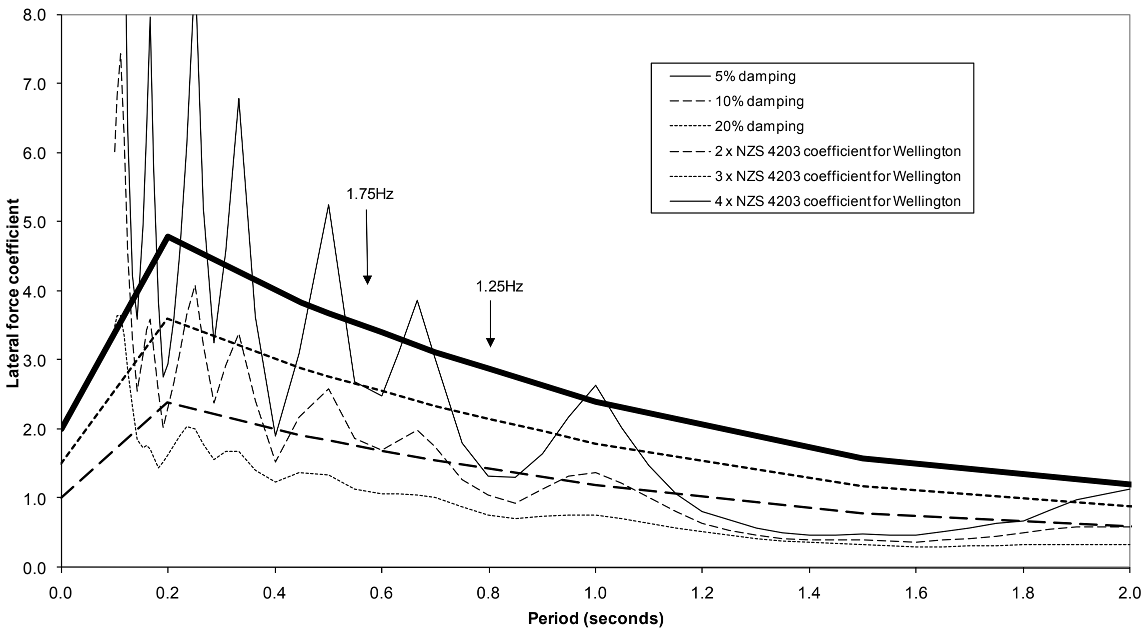


Figure 50. Summation of response spectra in Stages I and II

11. COMPUTER MODEL

11.1 Introduction

The theoretical relationship between load in the brick veneer versus the ceiling deflection for monotonic loading was calculated using the theory by Thurston and

Beattie (2008a), and for the test room it is the backbone curve shown as a bold line in Figure 51.

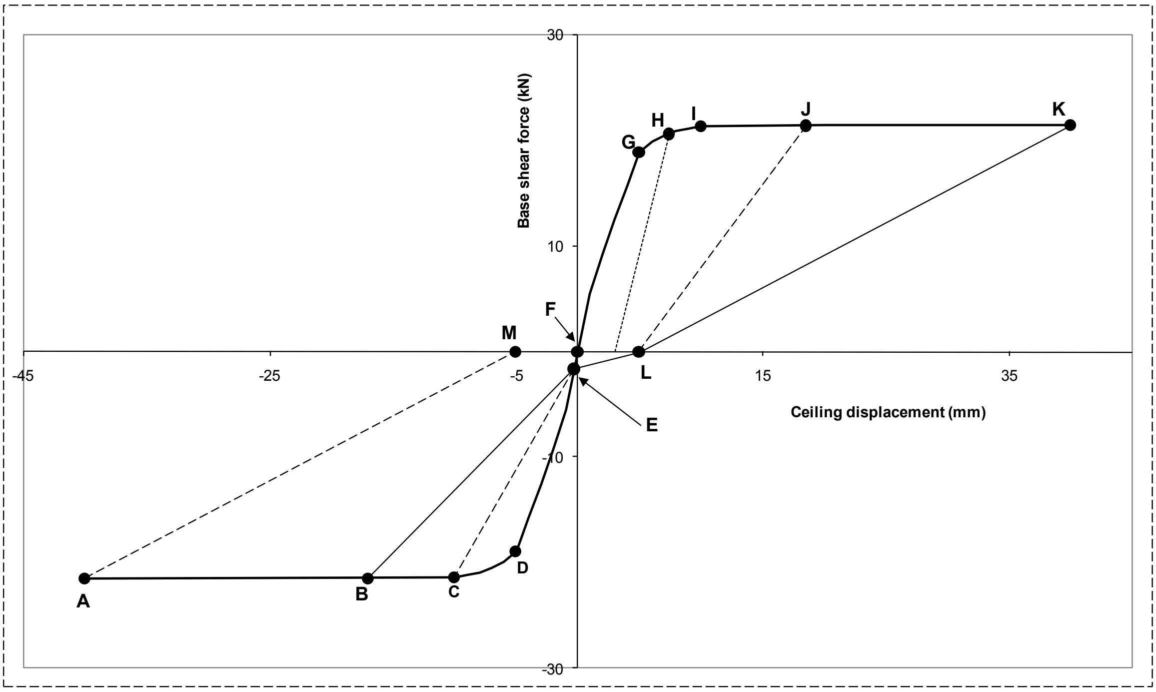


Figure 51. Hysteresis shape used in the computer model for predicting brick veneer panel response

The room was considered to be a SDOF oscillator with a mass on top, as shown in Figure 52. This mass was taken as the mass corresponding to the effective weight, W_T, at the top of the room. W_T was taken to be half the estimated weight of the walls (LTF and veneer) plus the room ceiling weight, W_c, calculated in Section 4.3, plus the added 34.88 kN weight of the concrete slabs. W_T was calculated to be 58.3 kN.

Appendix A describes a computer model of the SDOF system shown in Figure 52(a) to simulate the dynamic loading of the test room brick veneer. The properties of the spring are based on the Thurston and Beattie (2008a) theory, but now extend to the hysteretic shape shown in Figure 51. The program code is listed in Appendix A. To simulate the energy lost when the rocking veneer impacts the foundation, the horizontal rocking velocity is factored by "e" at each impact. To give best agreement with test results a value of e = 0.77 was chosen and was used in all analysis reported here. Damping was assumed to be 5% of critical.

In this section the model is verified by comparing the predictions with the shake table test results. It is then used to predict the response of the room when subjected to design earthquakes for Wellington corresponding to design spectra in NZS 4203 and NZSS 1170.5 (SNZ 2004). The purpose was to evaluate how the degree of shaking imposed on the test room corresponded to design level shaking. This is discussed further in Section 12.

11.2 Comparison of model predictions and test results

The computer model assumes the veneer and also the tie connection to the LTF are in an undamaged state at the commencement of any simulation. It ignores the influence of the linings (i.e. assumes they have become ineffective). The influence of the linings could simply be included by adding a separate element in parallel to simulate lining behaviour, as shown in Figure 52(b). However, this was not done here as the objective was to model the performance of the room at large deflections when the lining had become ineffective. All the comparisons given below are for this condition.

Figure 53 compares the model prediction of ceiling displacements with measurements made in Test T20 where the excitation was at 2 Hz. Agreement is good after the first few seconds of applied shaking, except that the model slightly overestimates the ceiling displacement. Good agreement in the early stages cannot be expected as the model assumes the brick tie lateral response has not softened, whereas prior testing has in fact softened the tie lateral resistance in the room.

A similar result was obtained for Test T21 at 1.5 Hz excitation (Figure 54). At higher frequencies of 4 Hz in Test T24 (Figure 55) and 9 Hz in Test T29 (Figure 56) agreement is still good.

Agreement deteriorates in Test T30 at 1.5 Hz (Figure 57). This is attributed to the ties becoming significantly weakened (many exhibited fatigue cracks but none were noted to have fully fractured). The agreement was far worse for Test T31 which was excited at 1 Hz (Figure 58). Many ties fractured in this test as discussed in Section 5.1 and Panel F stopped rocking part-way through the test (i.e. it contributed little to the lateral resistance) because of the fractured ties. To make some allowance for this, the model was modified simply by factoring the lateral resistance provided by the veneer by 0.8 and the model was re-run. The results in Figure 59 now show moderate agreement.

Results from Test T31 indicate that the response of the brickwork is very sensitive to the relationship between magnitude of shaking and model strength and small changes of magnitude/strength may result in large changes in the prediction of rocking displacement. This indicates that conservative design procedures will result in brickwork having low deflections in design level earthquakes.

11.3 Prediction of room response to NZS 4203 design spectra earthquakes

The computer model of Figure 52 was used to predict the response of the test room for "design level" earthquakes. The suite of earthquake acceleration records used in this prediction were provided by Davidson (2000) and were based on historic earthquake measurements "massaged" so that the spectra matched the NZS 4203 design spectra for intermediate soils given in Figure 4.6.1(b) of NZS 4203:1992. The match is excellent, as shown in Figure 60. Davidson's records were then scaled as required by Section 4.6.2.9 (b)(ii) of NZS 4203 for the numerical integration time history method using the following equation:

$C(T) = S_{m1} \times Ch(T,1) \times S_p \times R \times Z \times Lu$ ....(Equation 11-1)

where the Risk factor, R, and the Limit State Factor, Lu = 1.0, and the Zone factor for Wellington, Z = 1.2. S_p is assumed to be 1.0 and S_{m1} = 1.0, because the structure is responding elastically.

Figure 61 shows the predicted room deflection for computer runs where the Wellington level earthquakes were factored by an additional factor, F2. The Parkfield earthquake shows far greater deflections than the other records, and is therefore rejected because it is not consistent with the other records. Figure 61 indicates that F2 needs to be greater than 2.0 to provide the ceiling deflections achieved in the test. It also indicates that F2 = 1.0 results in deflections of approximately 20 mm.

Figure 63 shows an example hysteresis record for one analysis.

11.4 Prediction of room response to NZS 1170.5 design spectra earthquakes

Geological and Nuclear Sciences (GNS) supplied BRANZ with appropriate earthquake records for NZS 3604 type buildings. They were factored by k1 and k2 (also provided by GNS and reproduced in Table 3) to give a close fit to the design spectra, C(T), obtained from NZS 1170.5 (SNZ 2004) using the following equation:

$C(T) = C_h(T) \times Z \times R \times N(D,T)$ ……. (Equation 11-2)

These factors have different meanings to NZS 4203 use.

The Hazard factor, Z, is 0.4 for Wellington and thus differs from the Zone factors, Z, given in NZS 4203 and referred to in Section 11.3. The Return period factor, R, is taken as 1.0 for normal risk buildings. The Near Fault factor, N(D,T), is 1.0 for buildings with a natural period less than 1.5s, such as the NZS 3604 buildings considered in this study.

Equation 5.5(1) of NZS 1170.5 requires the target spectrum for time history analysis to be that given by Equation 11.2 above, but factored by (1+Sp)/2. Clause 4.4 of NZS 1170.5 specifies S_p to be 0.7 for buildings with a ductility greater than 2.0 (such as houses). With S_p = 0.7, this factor is (1+0.7)/2 = 0.85.

Figure 62 compares the target response spectra used for the NZS 4203 and AS1170.5 time history analyses.

Table 3. Summary of k_1 and k_2 factors provided by GNS

Site condition	Earthquake record	Principal component	Scale factor k_1	Scale factor k_2
Shallow	Tabas, Tabas 1978	TABAS78_2	0.48	1.10
	La Union, Mexico 1985	LAUNION85_1	2.32	
	El Centro, Imperial Valley 1940	ELCENTRO40_2	1.39	
Deep	El Centro, Imperial Valley 1940	ELCENTRO40_2	1.63	1.02
	Joshua Tree, Landers 1992	JOSHUA92_2	1.99	
	Llolleo, Chile 1985	LLOLLEO85_2	0.79	

Figure 64 shows the predicted room displacement for computer runs where the GNS Wellington level earthquakes were factored by an additional factor, F2. There is a scatter of results and the NZS 1170.5 method requires the largest deflection to be used. Figure 64 indicates that F2 needs to be greater than 1.0 to provide the ceiling displacements achieved in the test. It also indicates that when F2 = 1.0 displacements of approximately 36 mm are predicted.

11.5 Summary

Currently NZS 3604 is based on NZS 4203. Hence, the computer model should be based on the spectra stipulated in NZS 4203. This indicated that it would require 2.0 times the design shaking to match the displacements measured in the tests if the veneer had the predicted strength. However, the design methodology (Thurston and Beattie 2008a) proposed that the veneer design strength = (predicted strength) x 0.7. Thus, the imposed sinusoidal shaking corresponded to 2/0.7 = 2.9 times the veneer design strength.

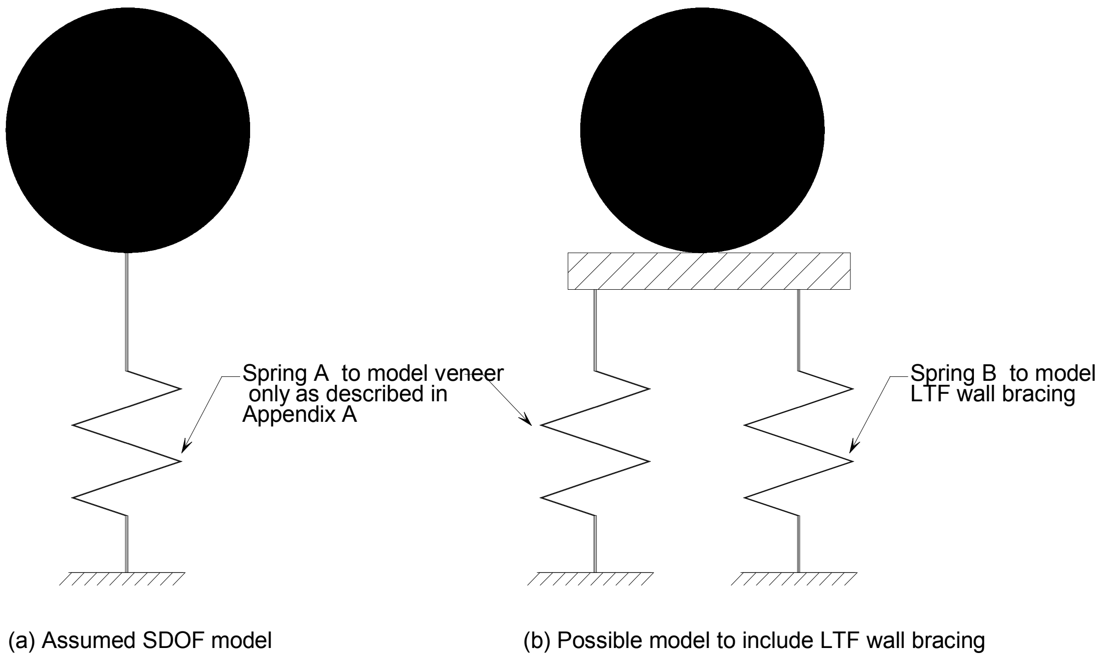


Figure 52. Schematic view of computer model

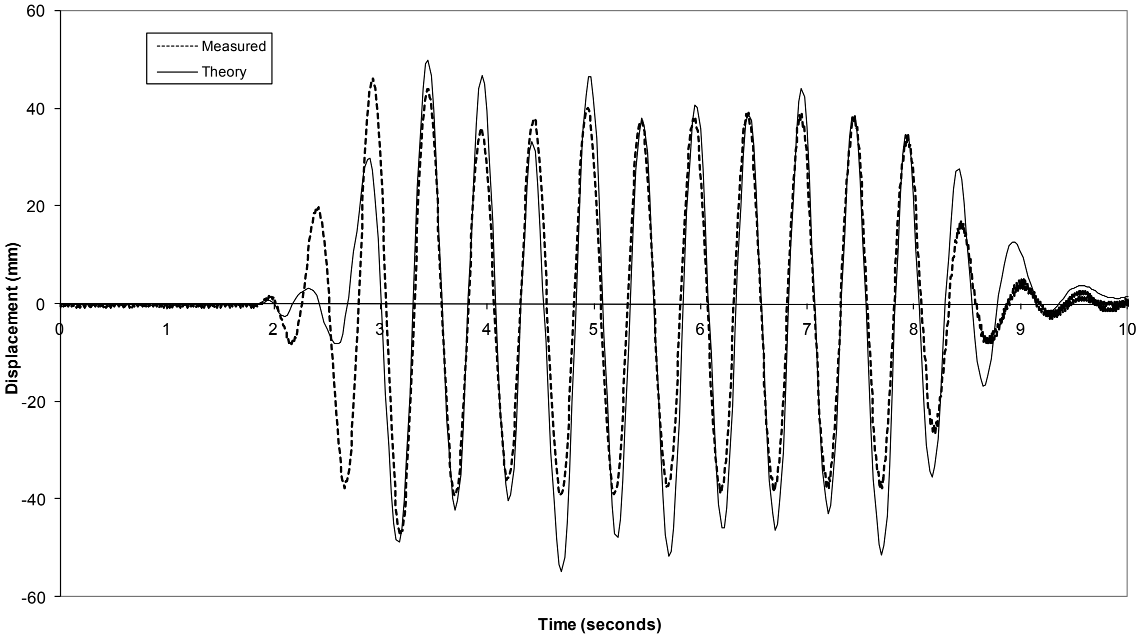


Figure 53. Comparison of measured and theoretical ceiling displacement time history for Test T20

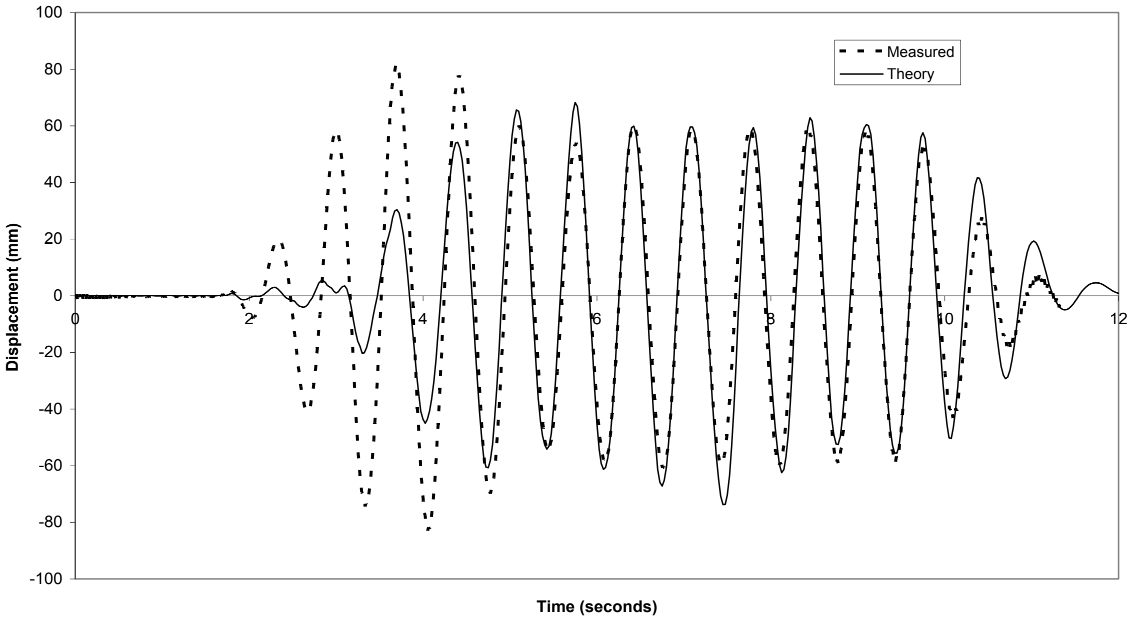


Figure 54. Comparison of measured and theoretical ceiling displacement time history for Test T21

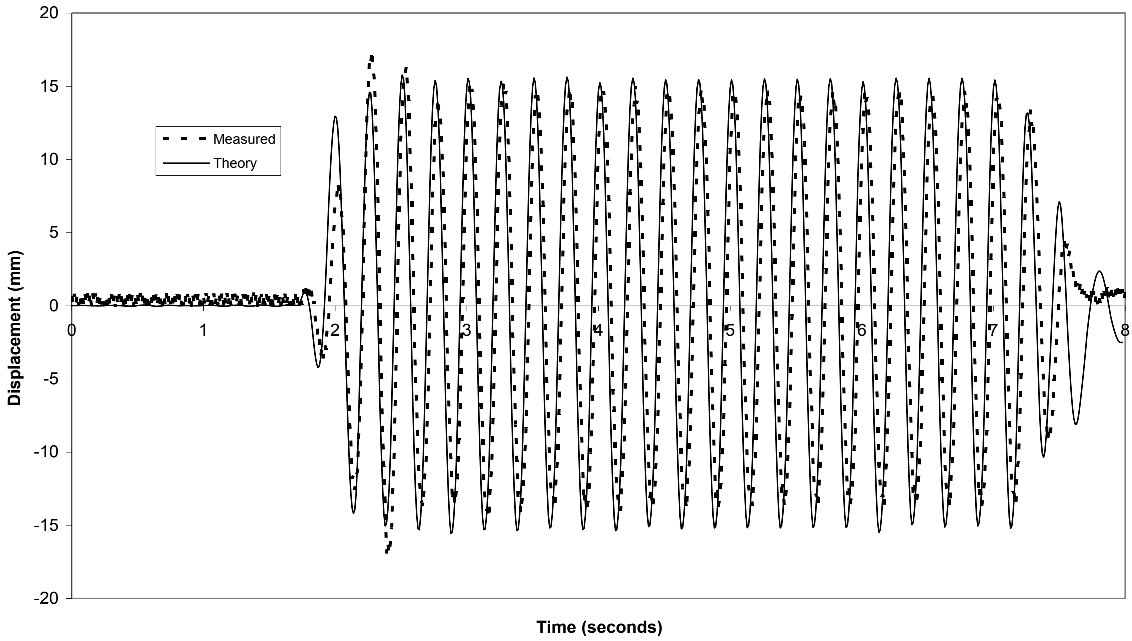


Figure 55. Comparison of measured and theoretical ceiling displacement time history for Test T24

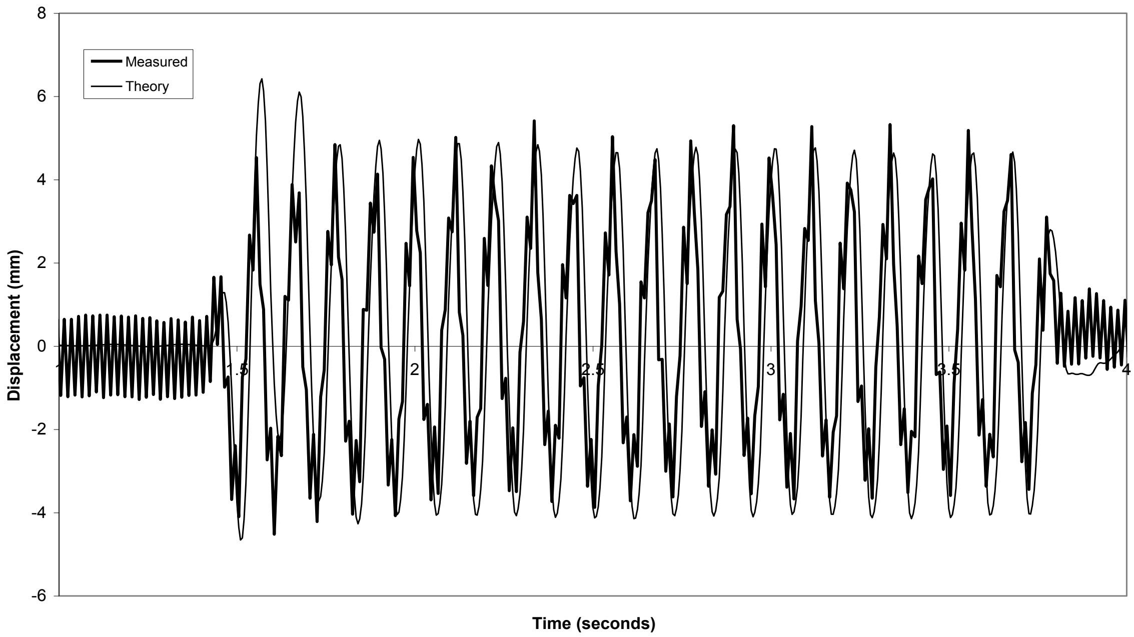


Figure 56. Comparison of measured and theoretical ceiling displacement time history for Test T29

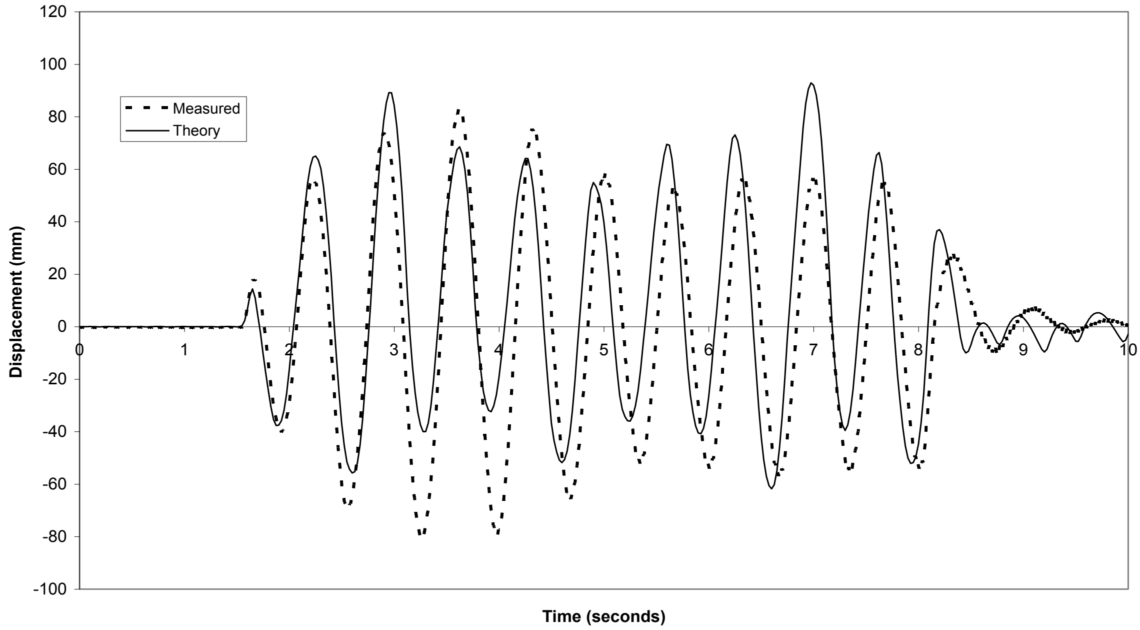


Figure 57. Comparison of measured and theoretical ceiling displacement time history for Test T30

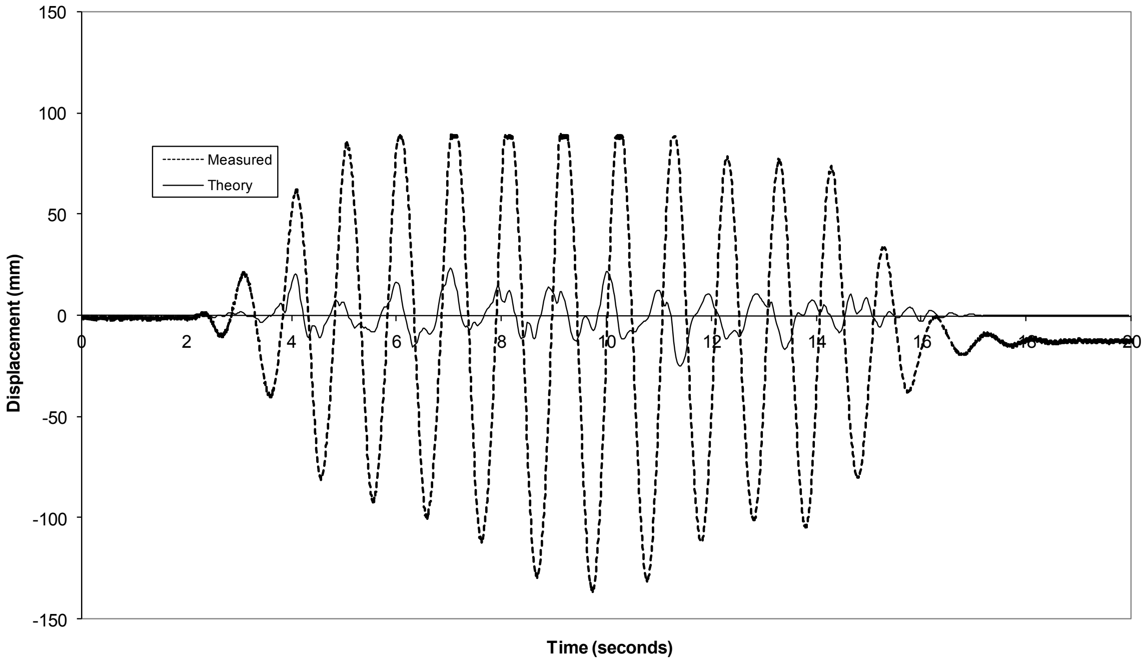


Figure 58. Comparison of measured and theoretical ceiling displacement time history for Test T31

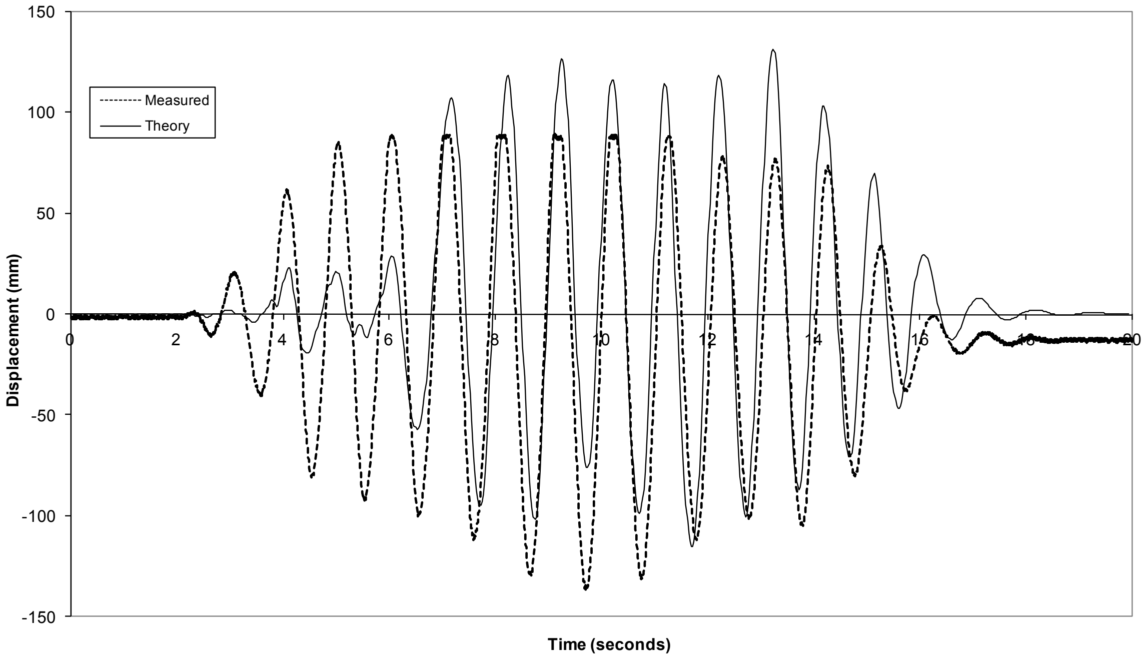


Figure 59. Comparison of measured and theoretical ceiling displacement time history for Test T31 with lateral resistance factored by 0.9

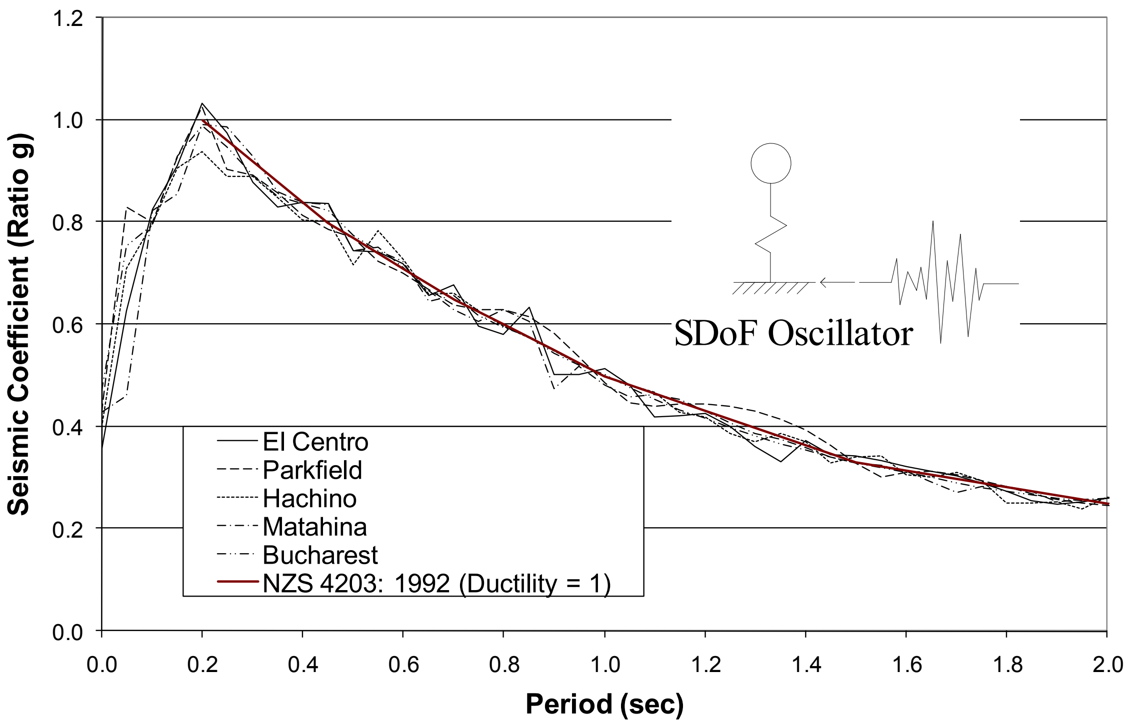


Figure 60. Comparison of acceleration response spectra from NZS 4203 and five earthquakes

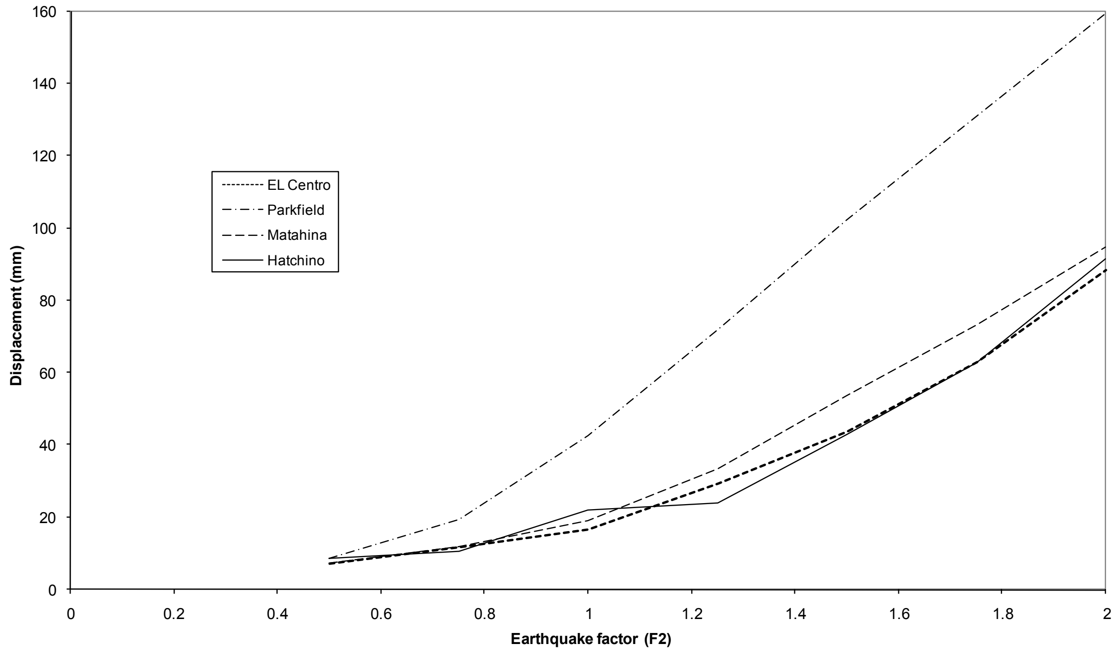


Figure 61. Predicted displacement of ceiling if room subjected to various earthquakes modified to fit the NZS 4203 Sp=1.0 Wellington design spectra

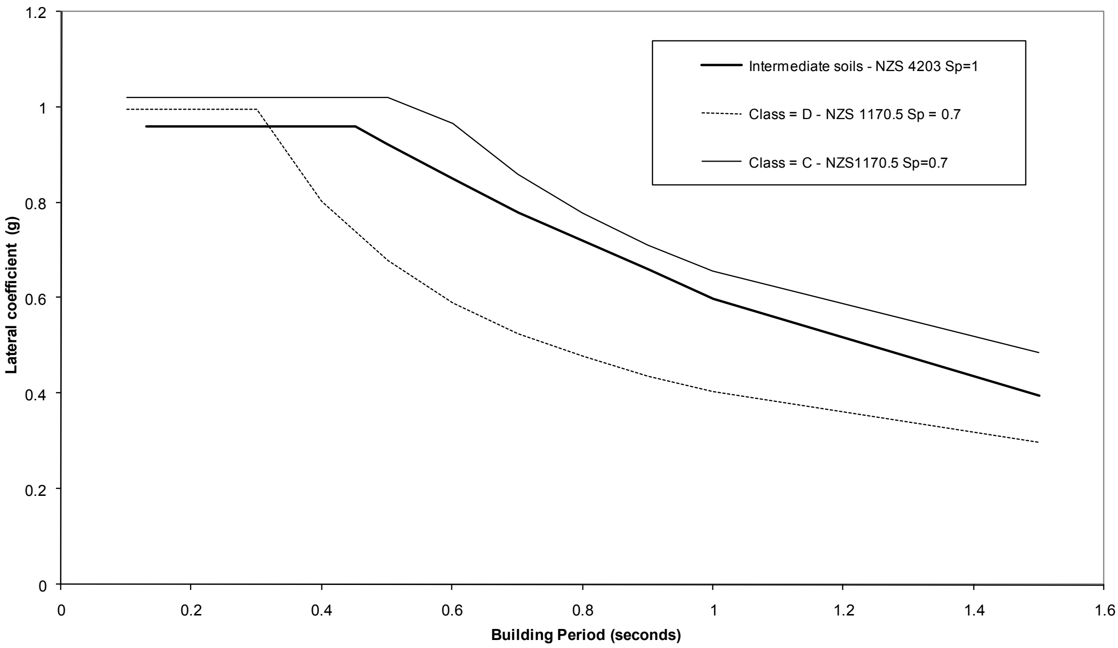


Figure 62. Comparison of target acceleration response spectra from NZS 4203 and NZS 1170.5 for matching earthquakes for time history analysis

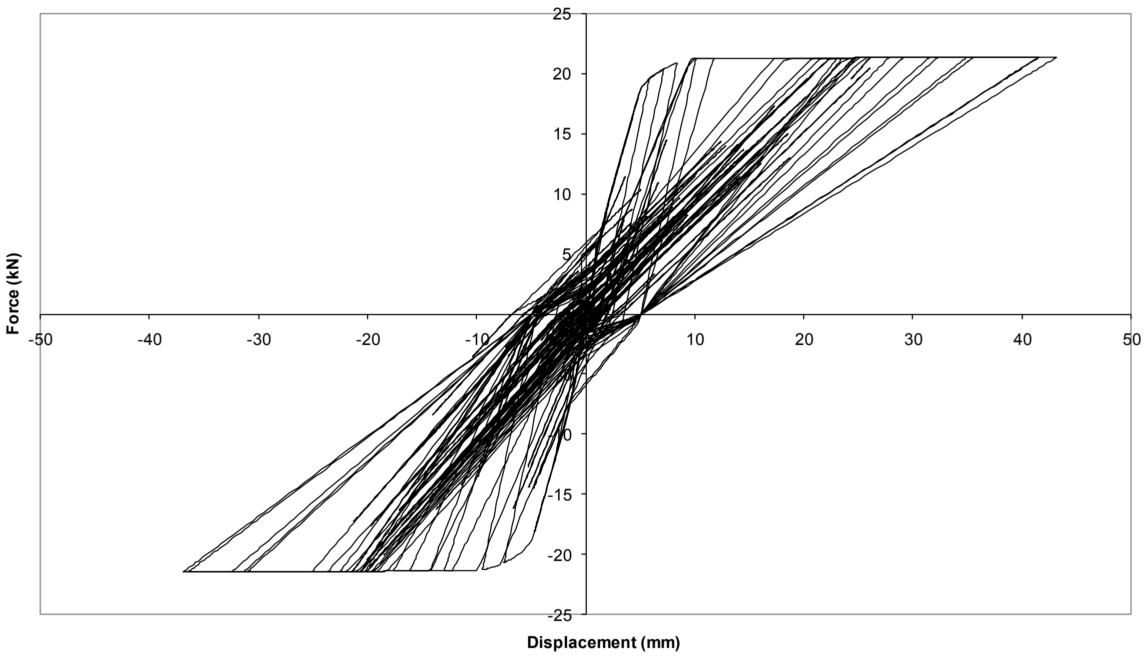


Figure 63. Total hysteresis from modified Hatchino earthquake with factor of 1.5

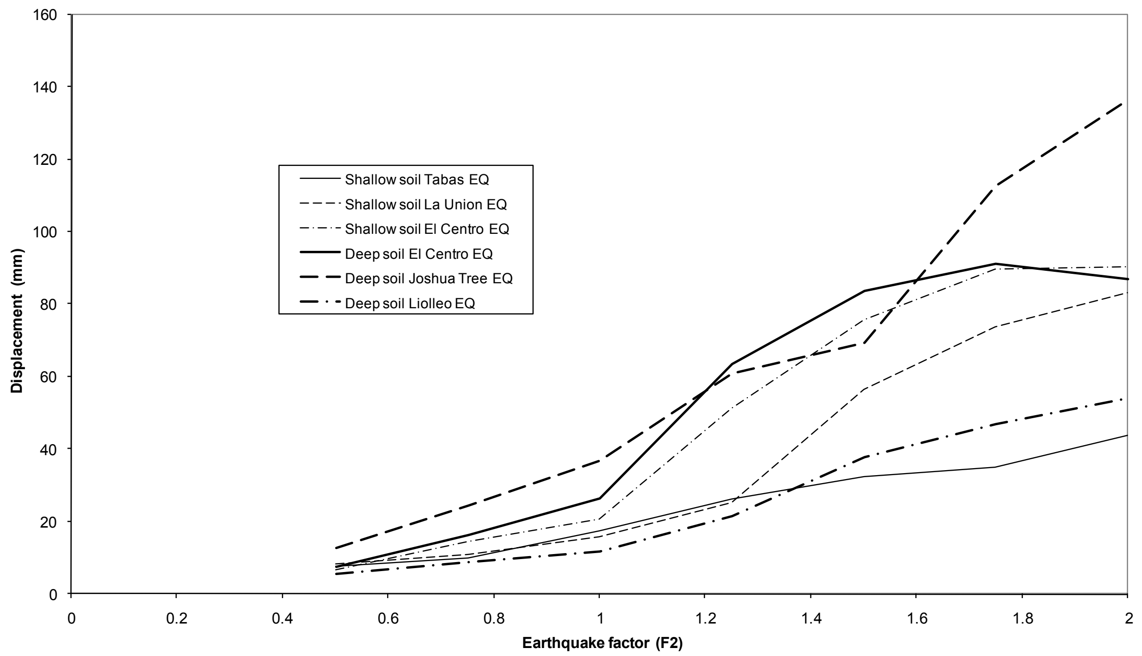


Figure 64. Predicted displacement of ceiling if room subjected to various earthquakes modified to fit the NZS 1170.5 S_p=0.7 Wellington design spectra

12. MAGNITUDE OF LOADING

When the test room was subjected to shaking, eventually the linings fell but the veneer stayed largely intact.

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 (i.e. the shaking needs to be related to the design strength of the veneer and LTF bracing walls, the NZS 3604 demand load for the mass used, the recorded table accelerations and the NZS 4203 design spectra).

Although the width of the LTF in the room was only 2.272 m, the mass placed on top increased the roof weight/unit length of the room to 6% more than that calculated for a 10 m wide house with a heavy roof.

12.1 Shaking with linings effectively detached

Once the linings no longer contributed to the bracing, the strength of the room was considered to be purely due to the lateral resistance of the brick veneer.

In Section 4.4, the ratio of the NZS 3604 lateral seismic load demand for Zone A for the room (and the design strength of the room) was 0.94 for the stage in the test when the linings had become ineffective as bracing.

In Section 10 base shear spectra were produced from the measured floor accelerations. By comparing these with the NZS 4203 design spectra at the natural frequency of the room, it was concluded that after the linings had become ineffective the veneer survived shaking at least 3.5 times the NZS 4203 Zone A design spectra in Stage I and 4.0 times the design spectra in Stage II.

Hence, it is concluded that had the veneer design strength been equal to the NZS 3604 lateral seismic load demand for Zone A, it could survive 4 x 0.94 = 3.76, say 3.5 times the design level earthquake. As the shaking effectively resulted in the strength of the veneer being fully utilised, it is concluded that the level of shaking corresponded to 3.5 times the design earthquake for Zone A.

To confirm the above methodology for assessing the effective value of shaking resisted, a computer model of the veneer was developed as described in Appendix A and discussed in Section 11. When displacements were computed using scaled earthquake records corresponding to the spectra stipulated in NZS 4203, it was found that it would require 2.0 times the design shaking to match the deflections measured in the tests if the room veneer had the predicted strength. However, the design methodology (Thurston and Beattie 2008a) proposed that the veneer design strength should be taken as 0.7 times the predicted strength. Thus, the earthquake shaking imposed corresponds to 2/0.7 = 2.9 times the design earthquake.

Note that the BRANZ P21 method assumes that the ULS displacement of the LTF walls is in the range of 16 to 36 mm. However, from **Error! Reference source not found.** the ceiling displacement relative to the table was ±82 mm in Test T30 and ±118 mm in Test T31. Based on the equal displacement rule, this implies a shaking intensity of approximately (118+82)/(16+36) = 3.8 times the design earthquake. Although this is very approximate, it does give a good correlation with the findings from the other two methods.

It is therefore concluded that the shaking effectively corresponded to approximately 3.5 times the design earthquake for Zone A after the linings had become ineffective.

12.2 Shaking with linings effectively attached

The linings, particularly in Stage II, contributed significantly to the room bracing.

In Section 4.4, the ratio of the NZS 3604 lateral seismic load demand for Zone A for the room and the design strength of the room when the linings were still effective as bracing were 0.56 and 0.33 for Stage I and II, respectively.

In Section 10, base shear spectra were produced from the measured floor accelerations. By comparing these with the NZS 4203 design spectra at the natural frequency of the room, it was concluded that while the linings were still effective the veneer survived shaking between 1.0 and 2.0 times the NZS 4203 ULS design shaking for houses. In Stage II, it was between 1.0 and 8.0 times the ULS shaking.

Hence, it is concluded that had the veneer design strength been equal to the NZS 3604 lateral seismic load demand for Zone A, then the test room with effective linings could survive between 0.56 x 1.0 = 0.56 and 0.33 x 8 = 2.64 times the design level earthquake. This range is too large to be useful.

The computer model discussed in Section 11, using the design veneer strength, calculated a maximum LTF wall deflection of 20 mm from 1.0 times earthquakes corresponding to the spectra stipulated in NZS 4203. However, the design methodology proposed that the veneer design strength should be taken as 0.7 times the predicted strength, which results in a design strength of 21.4 kN (from Section 4.3) x 0.7 = 15 kN. The LTF wall strength was estimated to be 10 kN in Stage I and 28 kN in Stage II (see Section 4.3). Thus, as a first approximation, the earthquake shaking imposed corresponds to 1.0 x (actual strength)/(design strength), i.e:

Stage I 1.0 x (21.4 + 10)/15 = 2.1 times the design earthquake.
Stage II 1.0 x (21.4 + 28)/15 = 3.3 times the design earthquake.

It is therefore concluded that the test shaking effectively corresponded to 3.3 times the design earthquake for Zone A to make the linings become ineffective.

13. SUMMARY

A clay brick veneer room was subjected to sinusoidal shaking. As the LTF lining became ineffective as a bracing element the deflections became large. The veneer panels which were loaded in-plane followed the ceiling displacement largely by rocking. The face-loaded portion of the veneer remained generally intact and moved with the end LTF walls. Near the end of the testing some ties developed fatigue cracks and some failed completely 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, with no slip being observed, and well attached to the LTF framing.

Two plasterboard lining constructions were used. In Stage I the lining was only lightly fixed and in Stage II it was fixed more strongly. The first damage to the lining in both Stages I and II occurred at 4 Hz shaking. Subsequent shaking at 2 Hz resulted in gradually increasing displacement amplitude from ±15 to approximately ±50 mm. Then shaking at 1.5 Hz gave displacement amplitudes of approximately ±82 mm. Finally, shaking at 1 Hz gave displacement amplitude of ±118 mm, but this large value may have been partially due to the effect of broken ties. The brick did not veneer collapse despite LTF deflection of ±118 mm.

Even after the extreme shaking imposed, which induced large veneer rocking deflections, once the shaking had stopped the veneer cracks almost closed and the brick veneer returned close to its original position. This is attributed to the rocking rather than sliding veneer deformation mechanism. The veneer could have been simply and cheaply repaired by re-pointing. In the final extreme test, a few bricks were dislodged by the pounding action as the rocking panels returned to their rest position. However, these too could simply be replaced. Damaged ties are unlikely to be an issue during real earthquake shaking except possibly after the severest shaking. It is considered that provided the veneer has not deflected by more than 25 mm, the ties can be considered to still work satisfactorily.

The shaking imposed was shown to be equivalent to approximately 3.3 times the NZS 4203 Zone A design earthquake prior to the linings becoming ineffective and 3.5 times the design earthquake after the linings had become ineffective.

A hysteretic computer model was developed based on the force versus deflection backbone theory described by Thurston and Beattie (2008a). Good agreement between the model and test results was obtained if the horizontal velocity of the rocking brick veneer was factored by 0.77 at each rocking panel impact. The model was used to predict the deflection-time history of the test room when subjected to design level earthquakes. This model may be used to predict house seismic response based on the brick veneer strength using the Thurston and Beattie theory and the LTF bracing strength determined separately.

14. CONCLUSIONS

It is concluded that a properly constructed house with clay brick veneer on all sides and incorporating resilient ties and bricks with vertical holes can be relied upon to resist design earthquake events with low displacements and little wall damage. The veneer is easily repairable. Collapse is unlikely under shaking several times the design event.

Brick veneer houses with significant bracing from the plasterboard lining will probably have a natural frequency greater than 5 Hz. If the building is founded on hard or intermediate soil, the high frequency component of an earthquake may well crack the veneer and cause lining damage. However, it is unlikely to also have sufficient low frequency energy content to make the veneer rock to large deflections. If the building is founded on soft soil the earthquake is unlikely to have sufficient high frequency energy to induce lining damage. Hence, the combination of lining and veneer bracing offers good seismic protection for the 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 face-loaded end wall ties prevented the “L-shaped corner veneer elements” from slipping along the base of the side walls. Thus, the side wall panels simply rocked in-plane with very little horizontal distortion of the ties (generally less than 10 mm).

(2) There was little out-of-plane differential movement between the end veneer panels and the LTF walls. The small amount that did occur (generally less than 10 mm) was due to compressive buckling of the ties. As predicted by Thurston and Beattie (2008a) this movement agreed with that in (1) above.

(3) There was good agreement between time history displacement measurements and computer model predictions. The hysteretic computer model was based on the backbone computer model developed by Thurston and Beattie (2008a).

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APPENDIX A. HYSTERETIC COMPUTER MODEL OF BRICK VENEER PERFORMANCE

A.1 Introduction

In this Appendix a computer model is explained that calculates the predicted displacement time history of the test room when excited by the measured table accelerations. Section 11 shows that this gives good agreement with experimental results, thus verifying the model. This model is then used to predict the displacement time history of the room when subjected to design level earthquake records. By this means, the design level earthquake record which the room could resist without exceeding acceptable deflections is determined, and hence the effective level of excitation that is imposed on the room.

A.2 Computer model

The theoretical relationship between the load in the brick veneer panel versus the ceiling deflection was calculated using the theory by Thurston and Beattie (2008a), and is shown as a bold line backbone curve ABCDEFGHIJK in Figure 65. This has been expanded to the full hysteresis loops shown in Figure 65 based on the measured hysteresis loops given by Thurston and Beattie (2008a) using the rules given below. Note that points L and M are where the rocking veneer returns to and impacts the foundation and the velocity is factored by a rebound factor 'e' discussed later.

Different parts of the hysteresis loop are governed by the rules discussed below and the code for the Excel Macro performing the analysis is also given below. The computer analysis processes in steps of 0.001s. If the analysis solution moves outside the current part of a loop in any step it migrates to the next appropriate part of the loop in the next step.

At each time step a new displacement is calculated and the force is calculated from the previous force + (current stiffness x displacement increment). The backbone force, called BackboneF, corresponding to the displacement is also calculated.

LoopCase 1. Part of hysteresis loop is DEFG. Points G and D are where the backbone curve is considered to deviate from a linear elastic performance and are set at +5 and -5 mm, respectively. Until the displacement is outside these limits the analysis assumes that the load displacement path is along DEFG. The slope of FG is called MaxStiffness.
If the displacement exceeds 5 mm the solution algorithm assumes LoopCase 2 for the next time step.
If the displacement is less than -5 mm the solution algorithm assumes LoopCase 5 for the next time step.

LoopCase 2. Part of hysteresis loop is GHIJK. Until the velocity becomes negative, the analysis assumes that the load displacement path is along GHIJK. When the velocity is negative the solution algorithm assumes LoopCase 3 for the next time step.

LoopCase 3. Part of hysteresis loop is KL or JL depending if the displacement corresponds to K or J when the velocity becomes negative. Point I is at +10 mm deflection. If the deflection is between G and I then the slope becomes "MaxStiffness" as shown by the dotted line from H.

The veneer is assumed to be dragging the LTF wall system via the ties and at its impact with the foundation it is assumed that the mass on the LTF is at a displacement of +5 mm. i.e. Point L is at a deflection of +5 mm and corresponds to the impact point of the brick veneer.

If the velocity is greater than 0 and the force is greater than BackboneF then the solution algorithm assumes LoopCase 2 for the next time step.

If the velocity is less than 0 and the force is negative then the solution algorithm assumes LoopCase 4 for the next time step.

LoopCase 4. Part of hysteresis loop is LE and its slope is rather arbitrarily set to half that in LoopCase 3 but not exceeding MaxStiffness/4.

If the velocity and force are greater than 0 then the solution algorithm assumes LoopCase 3 for the next time step with a stiffness = (3 x Previous LoopCase 3 stiffness + MaxStiffness)/4.

If the velocity is less than 0 and the force is less than BackboneF then the solution algorithm assumes LoopCase 5 for the next time step.

LoopCase 5, 6 and 7 correspond to LoopCase 2, 3 and 4 above but on the inverted mirror side of the hysteresis loop. LoadCase
+5 is ABCD, LoadCase 6 is AM, LoadCase 7 starts from M but is not drawn.

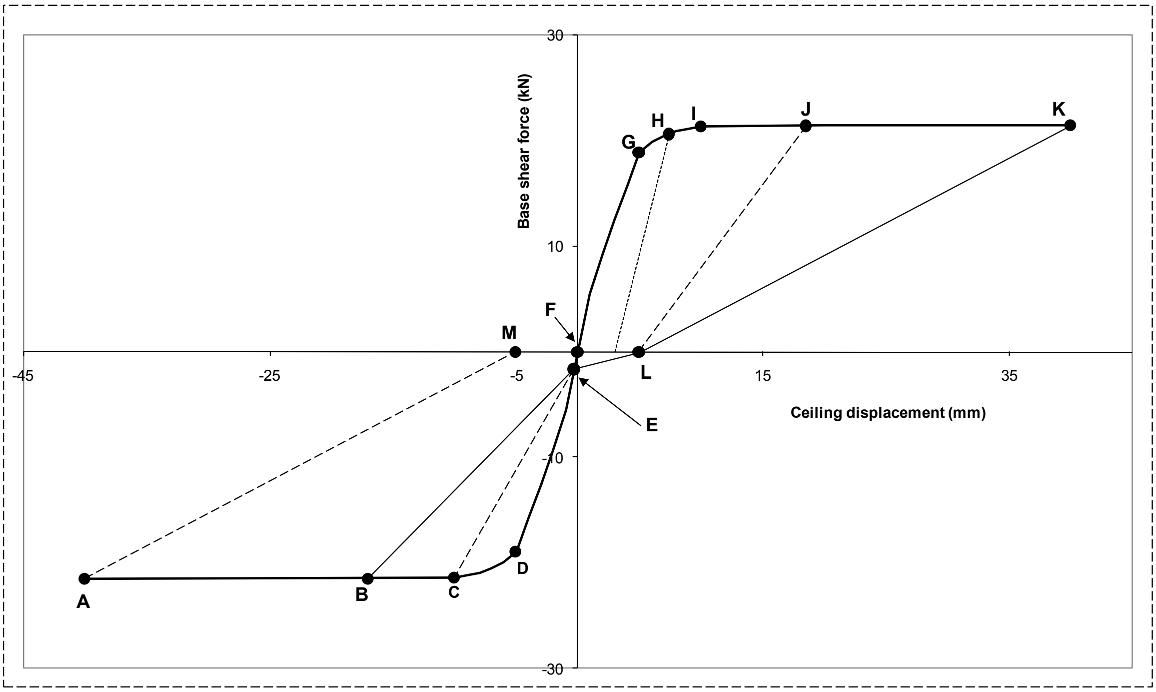

Figure 65. Hysteresis shape used in the computer model for predicting brick veneer panel response

Compute macro

```
Sub THanalysis()
'
' time history analysis by Stuart Thurston
'
'
Dim TableAcc As Double
Dim AccOld As Double
Dim GA As Double
Dim SpringK As Double
Dim NoOfIncrements As Long
Dim DampP As Double
Dim Damp As Double
Dim X As Double
Dim V As Double
Dim A As Double
Dim U As Double
Dim Amax As Double
Dim Xmax As Double
Dim Vmax As Double
Dim Mass As Double
Dim TimeInc As Double
Dim T As Double
Dim Time As Double
Dim OldTime As Double
Dim Coeff As Double
Dim BackboneK As Double
Dim BackboneF As Double
Dim Force As Double
Dim LoopCase As Single
Dim NewLoopCase As Single
Dim E As Double ' ratio velocity after to before impact - ie rebound velocity
Dim XOld As Double
Dim Limit1 As Double
Dim Limit2 As Double
Dim PeakForcePos As Double
Dim PeakForceNeg As Double
Dim Case3SpringK As Double
Dim Case6SpringK As Double
Dim Xlow As Double
Dim MaxStiffness As Double
Dim MinStiffnessPull As Double
Dim MinStiffnessPush As Double

Sheets("TimeHistory").Select
Range("E21:G2892").Select
Selection.ClearContents

MaxStiffness = Range("MaxStiffness") * 1.01
MinStiffnessPull = MaxStiffness
MinStiffnessPush = MaxStiffness

Range("Mess") = "Started macro"
E = Range("E")
```

```
NoOfIncrements = Range("NumInc")
TimeInc = Range("Inc") / CDbl(NoOfIncrements)
Mass = Range("BMass") ' Mass in Units tonnes
DampP = Range("DampRatio") ' Assume 5% damping
Limit1 = Range("Limit1")
Limit2 = Range("Limit2")
SpringK = Range("Springk")

'Initialisation
OldTime = 0#
AccOld = 0#
X = 0#
U = 0#
Xmax = 0#
Vmax = 0#
Amax = 0#
Damp = 2 * DampP * Sqr(SpringK * Mass) 'units tonnes/sec
Part = 1
LoopCase = 1
Force = 0#

Range("J21:K25000").Select
Selection.ClearContents


For j = 1 To 10000 'Look at each of up to 10000 lines of input data
  'check if all table accelerations have been processed
   Time = Range("DataV").Cells(j, 1)
   If Time < OldTime Then GoTo FinishedTH
   OldTime = Time
   'Read table accelerations from spreadsheet
   TableAcc = Range("DataV").Cells(j, 2) * 9810#
   For k = 1 To NoOfIncrements 'Subdivide acc interval into this number of increments

      GA = AccOld + (TableAcc - AccOld) * CDbl(k) / CDbl(NoOfIncrements)
      XOld = X
      A = GA - (Damp * U + Force) / Mass 'units mm/sec2
      V = U + A * TimeInc
      X = X + (V + U) / 2# * TimeInc
      If Abs(X) > Xmax Then Xmax = Abs(X)
      If Abs(V) > Vmax Then Vmax = Abs(V)
      If Abs(A) > Amax Then Amax = Abs(A)
      If X > 1000# Or X < -1000# Then GoTo AbortMe

      'Update Force from last deflection increment
      Force = Force + SpringK * (X - XOld)

      ' Get SpringK stiffness for next step
      'First get backbone properties to be used if needed.
      For i = 1 To 100
         If X < Range("Model").Cells(i, 1) Then GoTo GetStiffness
      Next i
      GoTo AbortMe
```

```
GetStiffness:

    BackboneK = Range("Model").Cells(i - 1, 3)
    ' Get force on backbone corresponding to X
    Xlow = Range("Model").Cells(i - 1, 1)
    BackboneF = Range("Model").Cells(i - 1, 2)
    BackboneF = BackboneF * 1000 + (X - Xlow) * BackboneK
    NewLoopCase = LoopCase


    If LoopCase = 1 Then
      SpringK = BackboneK
      Force = BackboneF
      If X > Limit1 And V > 0 Then NewLoopCase = 2
      If X < Limit2 And V < 0 Then NewLoopCase = 5

    ElseIf LoopCase = 2 Then
      SpringK = BackboneK
      Force = BackboneF
      If V < 0 Then
        NewLoopCase = 3
        SpringK = BackboneF / (X - Limit1)
        If X < 2 * Limit1 Then SpringK = MaxStiffness
        If MinStiffnessPush > SpringK Then MinStiffnessPush = SpringK
      End If
    ElseIf LoopCase = 5 Then
      SpringK = BackboneK
      Force = BackboneF
      If V > 0# Then
        NewLoopCase = 6
        SpringK = BackboneF / (X - Limit2)
        If X > 2 * Limit2 Then SpringK = MaxStiffness
        If MinStiffnessPull > SpringK Then MinStiffnessPull = SpringK
      End If
    ElseIf LoopCase = 3 Then
      If V > 0 And Force > BackboneF Then
        NewLoopCase = 2
        Force = BackboneF
        SpringK = BackboneK
      ElseIf V < 0# And Force < 0# Then
        V = V * E 'Impact
        NewLoopCase = 4
        Case3SpringK = SpringK
        SpringK = SpringK / 2#
        If SpringK > MaxStiffness / 4# Then SpringK = MaxStiffness / 4#
      End If
    ElseIf LoopCase = 6 Then
      If V < 0# And Force < BackboneF Then
        NewLoopCase = 5
        Force = BackboneF
        SpringK = BackboneK
      ElseIf V > 0# And Force > 0# Then
        V = V * E 'Impact
        NewLoopCase = 7
        Case6SpringK = SpringK
```

```
            SpringK = SpringK / 2#
            If SpringK > MaxStiffness / 4# Then SpringK = MaxStiffness / 4#
        End If
    ElseIf LoopCase = 4 Then
        If V > 0# And Force > 0# Then
            NewLoopCase = 3
            SpringK = (3 * Case3SpringK + MaxStiffness) / 4# 'NEW
        ElseIf V < 0# And BackboneF < Force Then
            NewLoopCase = 5
            SpringK = MinStiffnessPull * 1.5
            If SpringK > MaxStiffness / 2# Then SpringK = MaxStiffness / 2#
        End If
    ElseIf LoopCase = 7 Then
        If V < 0# And Force < 0# Then
            NewLoopCase = 5
            SpringK = (3 * Case6SpringK + MaxStiffness) / 4#
        ElseIf V > 0# And BackboneF > Force Then
            NewLoopCase = 3
            SpringK = MinStiffnessPush * 1.5
            If SpringK > MaxStiffness / 2# Then SpringK = MaxStiffness / 2#

        End If
    End If

    U = V
    LoopCase = NewLoopCase
    Range("Hyst").Cells(j, 1) = X
    Range("Hyst").Cells(j, 2) = Force / 1000#

Next k
AccOld = TableAcc
'write out currrent values to spreadsheet
Range("PredCeilDisp").Cells(j, 2) = X
Range("PredCeilDisp").Cells(j, 3) = V
Range("PredCeilDisp").Cells(j, 4) = A / 9810#
Range("PredCeilDisp").Cells(j, 5) = Force / 1000#

 Next j

 FinishedTH:
' Print out TH results
    Range("MaxVals").Cells(1, 1) = Xmax
    Range("MaxVals").Cells(1, 2) = Vmax
    Range("MaxVals").Cells(1, 3) = Amax / 9810#

  GoTo ExitNow
    AbortMe:
    Range("Mess") = "aborted!!"
    ExitNow:
    End Sub
```