Seismic performance of New Zealand two-storey brick veneer houses

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ABSTRACT: Recent tests at BRANZ have shown that modern, clay-brick veneer single-storey houses perform well under seismic loading and the veneer can be relied upon to resist a significant portion of the seismic load. At lateral in-plane displacements well in excess of those expected in a design level earthquake the veneer settled back close to its original position on unload and cracks almost closed and could easily be repaired. Partial or full collapse did not occur even at very large lateral deflections. Tests by others have shown that modern clay brick veneer construction should perform well under out-of-plane earthquake loading.

This paper describes slow cyclic racking tests on a 6.7 m x 3.9 m plan two-storey homogeneous brick veneer house with window and door openings. Two-storey construction is currently not allowed by NZS 3604 and requires specific design. At design level displacements the veneer cracks mostly closed in a similar manner to the single-storey construction. The upper-storey veneer performed in a similar manner to a single-storey veneer but the lower-storey veneer was far stiffer. The veneer generally remained firmly attached to the timber framing even when the roof and first floor were displaced ±143 mm and ±69 mm respectively, although cracking at this stage was severe and did not completely close. Proposals to allow the veneer to provide a bracing function in NZS 3604 structures and to allow suitably proportioned two-storey brick veneer to be used without specific design will be submitted to Standards New Zealand.

1 INTRODUCTION

Historically brick veneer houses have not performed well in earthquakes. However, recent tests at BRANZ (Thurston and Beattie, 2008a,b) have shown that modern single-storey construction performed excellently at well past the design level displacements (with provisos). This was attributed to the use of better brick-ties which are screwed to studs and the use of bricks with internal holes.

NZS 3604 (SNZ, 1999) places a height restriction of 4 m on masonry veneer tied to timber framing. Thus, without specific design its application is effectively limited to single storey or the lower of two storeys. The study reported here investigates whether this restriction can be relaxed.

NZS 3604 assumes that the veneer itself provides no bracing function. The veneer seismic inertia forces are assumed to be transferred via the brick ties acting as axial elements (face-loading) or shear elements (in-plane loading) to light timber-framed (LTF) bracing walls. However, BRANZ tests (Thurston and Beattie, 2008a) have shown that single-storey veneer can carry large bracing forces in an earthquake when the ties will transfer in-plane shear load from the LTF to the veneer. A theory to calculate the shear load carried by the veneer was developed and the prediction gave good agreement with test measurements. The current study investigates whether two-storey veneer can also provide a bracing function.

Brick ties must now be screwed to the timber frame as it was found that when nailing a tie to a stud, the stud vibrated and nearby ties loosened in the partially set mortar (Shelton, 1996). To ensure brick ties can resist the expected loads, ties must nowadays be shown to be able to resist a stud shrinkage movement of 10 mm, four cycles of seismic displacement of ±20 mm in the plane of the veneer and
then have an out-of-plane strength of at least the weight of the tributary area of veneer factored by 1.74 (Zone A), 1.45 (Zone B) and 1.18 (Zone C). Thus, failure of either the ties or their connections is not expected in the body of the veneer in a design earthquake, although the portions of veneer cantilevered past the last tie at tops or ends of walls or at openings need to be shown to be safe.

2 BRANZ BRICK VENEER SEISMIC TESTS

In Phase 1 (Thurston and Beattie, 2008a) slow cyclic racking tests were performed on two full-scale brick-veneer single-storey “rooms” which had both window and door openings. At displacements well in excess of that expected in design level earthquakes, the veneer cracks closed on unload and the building suffered little veneer damage.

In Phases 2 and 3 (Thurston and Beattie, 2008b,c) shake table tests were performed on a clay-brick veneer clad room and a concrete-brick veneer clad room, respectively. These used an added mass to simulate roof loads. Sinusoidal motion was applied to the shake table. The shaking frequencies were selected to excite the in-plane loaded veneer rather than the face-loaded veneer. Building performance at well in excess of design loading was excellent.

This paper describes cyclic racking of a two-storey brick veneer building (Phase 4). Full details are described elsewhere (Thurston and Beattie, 2009).

BRANZ is currently undertaking shake table tests on brick veneer walls in the out-of-plane direction after the walls had been first subjected to in-plane displacements.

3 OTHER BRICK VENEER TESTS

Little testing has been performed on the in-plane seismic performance of brick veneer tied to LTF. What has been done is summarised by Thurston and Beattie (2008a). Shake table tests have recently been performed on a 6 m x 6 m single-storey building at the University of California, San Diego using ties commonly used within USA that are weaker when nailed and less flexible when screwed than used in NZ. This clearly demonstrated that ties screwed to the LTF were far more effective than when nailed (B. Shing, Personal communication). An approximately 2.7 m square building using steel framing will be tested in Melbourne shortly. Neither of these has been reported at the time of writing. Significantly more testing has been performed in the out-of-plane direction.

4 BRICKS, BRICK-TIES AND MORTAR USED IN BRANZ TESTING

The bricks used in the BRANZ tests had dimensions 230 mm long x 76 high x 70 mm wide. They had five vertical holes, of cross-section 32 x 23 mm, for the full brick depth, mainly to lighten the brick and aid the kiln-drying process. When the bricks were being laid, the holes partially filled with mortar as shown in Figure 1, thus effectively forming mortar dowels, which greatly enhances the horizontal shear strength between bricks. In the latter stages of the BRANZ two-storey test the mortar dowels did shear off at some locations and some portions of brick veneer consequently slid along horizontal mortar cracks between bricks. However, generally this effect was small.

The bricks were laid by tradesmen using pre-bagged dry mortar. Approximately 10 mm thick mortar joints were used between the bricks on both horizontal and vertical joints. This had a measured 28 day crushing strength of 20 MPa. The required strength is only specified to “follow the requirements of the masonry suppliers” (SNZ, 2001) which is generally taken to be 12.5 MPa. Recent mortar compression tests on self-mixed mortar randomly taken from 10 Auckland construction sites gave a wide range of strengths and an average strength of 7.1 MPa (privately loaned report). BRANZ will shortly begin an investigation into the relationship between veneer seismic performance and mortar strength. Pre-bagged dry mortar has many advantages including good workability, consistent colour and strength, less efflorescence, less site wastage, and consequently less rework (Oliver, 2006). Both Oliver and BRANZ consider that the advantages of pre-bagged dry mortar far outweigh the additional materials cost.
Commonly available 85 mm long, hot-dipped galvanised, brick-ties were dry bedded onto the 70 series bricks, rather than being fully encapsulated within the mortar, at spacings of 340 mm vertically and 600 mm horizontally. Ties were secured to the face of the timber studs using 35 mm long galvanised, self-drilling Tek screws which had a 4.4 mm diameter shank and a 14 mm diameter washer formed as part of a hexagonal-shaped head.

Figure 1. Holes in bricks. Mortar formed dowels between the bricks

5 BRANZ CYCLIC RACKING TEST ON A TWO-STOREY BRICK VENEER BUILDING

A two-storey brick veneer building with plan dimensions 6.7 m x 3.9 m was cyclically racked to increasing displacement levels cumulating in a roof and first floor displacement of ±143 mm and ±69 mm respectively. The slow speed loading did not induce the inertia forces on the veneer which would occur in real earthquakes and thus the test did not test veneer out-of-plane performance but is expected to give a reasonable simulation of in-plane performance.

The veneer was separately supported on beams on rollers which enabled the shear load carried at the base of the veneer to be directly measured. Large veneer crack widths (Figures 2 and 3) occurred at the maximum displacements. This is far in excess of the design level seismic displacement (approximately 24 mm per storey for plasterboard lined houses). However, all ties remained embedded in the brick mortar and firmly attached to the LTF at test completion.

Three bricks fell but generally the veneer remained firmly attached to the framing throughout the testing. Vertical cracks occurred near building corners in the latter stages of the tests but the veneer side and end wall panels still acted compositely. Some of the cracks at this stage cut through the bricks themselves rather than purely following the mortar courses and some of the residual crack widths near the end of the testing would be unacceptable to an owner. Thus, the entire building would have likely needed re-cladding.

Initially the test was performed under load control with the applied forces at the roof being twice that at the first floor. This complied with the lateral force distribution assumed in NZS 3604. However, after the top-storey shear strength had been exceeded there was little cracking in the bottom storey and the first floor displacement was low. Hence, to induce damage in the lower-storey the loading was changed to deflection control with the displacement of the first floor being set to be approximately half that at the roof.

Sometimes cracks which were clearly visible at peak loads in one stage closed and could not be seen at peak loads in subsequent stages, particularly if other nearby cracks had formed.
Only a few veneer cracks occurred for absolute roof displacements up to ±30 mm and these were narrow at peak loads and closed to become invisible to the naked eye at unload, except that cracks in the top lintels could still easily be seen.

The major deformation mechanism for absolute roof displacements up to ±46 mm was rocking of the upper-storey panels about cracks emanating from window corners. These were mainly horizontal cracks on Side 1 and diagonal cracks in the first floor spandrel panels on Side 2. The horizontal cracks closed to be difficult to see at unload but the vertical and stepped cracks could often be seen without difficulty.

![Figure 2. Veneer cracking on Side 1 at peak imposed displacement](image)

At absolute roof displacements up to ±64 mm damage was largely confined to the upper storey. This is not surprising as the peak first-storey floor deflection was only ±12 mm, even though the veneer carried high shear loads. The cracks in the lower-storey veneer were very narrow.

The test regime then changed to deflection control. Many new cracks subsequently occurred in the first floor spandrel, emanating from the top corners of the lower-storey windows. Brick crushing at the
bottom corners of the bottom windows occurred due to the high axial stress at these locations as the panels above rocked, pivoting at these corners.

Measurements were taken of the in-plane and out-of-plane slip of the veneer relative to the LTF. The in-plane slip was only approximately 20% of the LTF movement at corresponding heights. The out-of-plane movement of the veneer relative to the LTF was less than ±3 mm along the building sides. On the end walls the veneer tended to move away from the LTF (by a process of the “L”-shaped ties straightening) but this was generally less than 6 mm.

Upward movement of the veneer was measured at 128 locations for the three stages of test. These were used to plot panel rotations against height up the panel and these confirmed that most of the panel rotation occurred at the cracks emanating from the bottom corners of top windows. It was only in the latter stages of test that panel rotations at the cracks emanating from the bottom corners of bottom windows were large. The rotations were used to derive veneer horizontal deflections and this showed that most of the veneer displacement was due to rotation of the veneer rather than shear slip or other forms of deformation. Shear slip was observed along some upper-storey cracks in the latter stages of the test. Inspection revealed that mortar dowels had been sheared off in these locations.

It appears that two-storey brick veneer with construction similar to that tested can sustain design level in-plane displacements with only minor damage to the veneer. Only a few bricks are expected to fall at twice the design level displacements. The brick veneer will resist high in-plane seismic shear forces which will result in building displacements being far lower than design level displacements calculated ignoring the veneer shear stiffness contribution.

A plot of the force applied at the roof versus the displacement of the roof relative to the first floor is shown in Figure 4. This applied load would have been partially resisted by the lined LTF with the remainder being carried by the veneer. As the upper-storey lining was only screwed at 600 mm centres and vertical joints were not stopped, the upper-storey LTF bracing strength is expected to be low, particularly at large displacements when the lining fixings had effectively failed.

A plot of the shear force directly measured at the base of the veneer versus displacement of the first floor is shown in Figure 5.

![Figure 4](image-url)  
*Figure 4. Roof load versus upper-storey displacement hysteresis loops for the two-storey building. Proposed veneer design bracing strength and predicted strength are also shown*
6 THEORY FOR VENEER WALLS WITH TOP SPANDRELS

Thurston and Beattie (2008a, b) developed a method for calculating the load carried by single-storey veneer with no veneer lintels above the windows. They extended this (Thurston and Beattie, 2009) to include the weight of brickwork above the piers. It was found that the weight of the veneer above significantly increased the theoretical bracing strength of the veneer, especially for the lower-storey veneer.

The two-storey building in the test described in this paper can be analysed as two single-storey buildings, one on top of the other. The results of the analyses are compared with the measured strength in Figures 4 and 5. The prediction is slightly lower than that measured for the upper storey. However, as the applied load shown in Figure 3 includes shear carried by the lined LTF (whereas the model only predicts the shear force in the veneer), all that can be said is the prediction is reasonable. The prediction is unconservative for the lower storey where the test measurement in this case is the shear directly carried by the veneer. It is therefore considered that cracking of the spandrels resulted in a weaker failure mechanism than the simple pier rocking mechanism assumed for the lower storey.

7 PROPOSED BRICK VENEER DESIGN BRACING STRENGTH

A conservative method to calculate the bracing resistance provided by brick veneer is derived below for both single and two-storey buildings. Thurston and Beattie (2008a) showed that shear slip and pier rocking would not occur at this bracing resistance. It is proposed that this be incorporated into the next revision of NZS 3604. It is only to be applicable where bricks were similar to that tested. That is:

- The mortar has a compressive strength of at least 12.5 MPa
- The bricks have a height-to-length ratio no greater than 0.5
- The bricks have full depth holes comprising not less than 12% of the horizontal cross-sectional area and with a minimum dimension of 20 mm.
7.1 Proposed veneer design bracing strength for single-storey construction

Consider a brick veneer wall of length L tied to LTF with stud height H. It is proposed that the design seismic shear that can be carried by the veneer, V1, is the self-weight seismic inertia force assumed by NZS 3604 to be transferred to the top of the adjacent LTF. Shelton (2006) showed that V1 is given by:

\[ V_1 = 0.5C \rho' L H \quad \ldots (1) \]

Where:
\( C \) = lateral force coefficient
\( \rho' \) = average veneer weight per unit area (reduced to include the effect of openings) (kN/m²)
\[ = \rho \left\{ \frac{\text{area of wall not including openings}}{\text{gross area of wall, including openings}} \right\} \]
\( \rho \) = unit weight of veneer (kN/m²).

Shelton (2006) stated that: \( C = 0.241 \) for Zone A, \( 0.181 \) for Zone B and \( 0.121 \) for Zone C and \( \rho = 2.2 \) kN/m² for veneer and that 30% of the wall area is assumed to be openings.

Rather than forces in kN, NZS 3604 refers to forces as BU’s, where \( 20 \text{ BU} = 1 \text{ kN} \). Incorporating these values the veneer design bracing strength can be expressed as:

\[ V_1 = KLH \quad \ldots (2) \]

Where \( K = 0.5C \rho' \) and is a constant given in Table 1. For example, for heavy-weight cladding in Zone A, \( K \) was calculated from \( K = 0.5 \times 0.241 \times 2.2 \times (1-0.3) \times 20 = 3.71 \).

Table 1. Value of K for Eqn. (2)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Zone A</th>
<th>Zone B</th>
<th>Zone C</th>
</tr>
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<tbody>
<tr>
<td>Zone A</td>
<td>3.71</td>
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<td></td>
</tr>
<tr>
<td>Zone B</td>
<td>2.78</td>
<td></td>
<td></td>
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<tr>
<td>Zone C</td>
<td>1.86</td>
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</table>

7.2 Comparison between proposed veneer design bracing strength and single-storey test result

Figure 6 provides a comparison of the proposed design shear bracing strength with the single-storey test results from Thurston and Beattie (2008a). As the veneer shear was measured at the bottom of the veneer, the design level is plotted as \( 2 \times V_1 \), as illustrated in Figure 7. Clearly this design method is very conservative. The shear carried by the veneer in the test came from load transfer from the LTF via the brick ties which is a mechanism conservatively ignored in the proposed method.

Figure 6. Comparison of calculated veneer design bracing strength and test results
7.3 **Proposed veneer design bracing strength for two-storey veneer construction**

It is proposed that a two-storey brick veneer house be considered as two single-storey brick veneer constructions, one on top of the other. Thus, the design bracing strength for the upper-storey veneer is calculated as per Section 7.1. Provided the openings in the lower storey do not exceed certain limits, it is proposed that the lower storey can be treated similarly except \( H \) in Eqn. 2 is now the height from the top of the concrete foundation beneath the veneer to the eaves.

7.4 **Comparison between proposed veneer design bracing strength and two-storey test results**

Figure 4 and Figure 5 give comparisons of the proposed bracing strengths with the upper and lower storey test results for the two-storey brick veneer building. Clearly the design method of Section 7.3 is very conservative for both storeys.

7.5 **Comparison between proposed veneer design bracing strength and NZS 3604 demand loads**

Thurston and Beattie (2009) compared the bracing ratings derived using the above method with the bracing demand stipulated in NZS 3604 for three building shapes with heavy and light roof and heavy wall cladding. The average, maximum and minimum percentages are shown in Table 2 for loading in the building long-side direction. (Internal walls are most effective in the other direction.) The percentages carried by the veneer are low but still useful. They are less for two-storey veneer, which is favourable as there is reason to be more conservative with two-storey applications.

<table>
<thead>
<tr>
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<th>Percentage of total earthquake load carried by brick veneer</th>
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<tbody>
<tr>
<td></td>
<td>Single storey</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td>29.2%</td>
</tr>
<tr>
<td><strong>Maximum</strong></td>
<td>48.5%</td>
</tr>
<tr>
<td><strong>Minimum</strong></td>
<td>18.8%</td>
</tr>
</tbody>
</table>

**Table 2. Comparison of proposed veneer bracing strength with demand loads from NZS 3604 for three building shapes (in long side direction)**

8 **CONCLUSIONS**

Brick veneer is far more resilient to earthquake loads in buildings using modern construction materials
and methods than was the case with historic construction. NZS 3604 can take advantage of this improvement of performance by allowing two-storey veneer construction within its scope and by attributing a bracing function to the veneer.

9 ACKNOWLEDGMENTS

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10 REFERENCES


