JAMES COOK CYCLONE STRUCTURAL TESTING STATION

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SIMULATED WIND LOADING OF A MELBOURNE STYLE BRICK VENEER HOUSE

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SYNOPSIS

Simulated wind loading tests were conducted on a house built to represent normal building practice in Melbourne. To ensure that the house was of typical construction timber was sent from Southern Victoria and a team of Melbourne builders was engaged to erect the frame.

The wind loads applied to the test house were based on the provisions of the Wind Loading Code for terrain category 3 (suburban) Melbourne wind conditions. Comparisons were made with the code requirements for Sydney and Brisbane. Large steel frames, hydraulic rams and load spreading devices were used to apply the simulated uplift and racking pressures equivalent to those generated by a severe wind storm. The response of the house was measured by electronic displacement gauges and fed into a portable computer used to store the data and process it later.

Under combined uplift and lateral wind loading the house was easily able to resist the simulated average design wind pressures for suburban Melbourne, Sydney or Brisbane. The test house also resisted pressures equivalent to those likely to occur in terrain category 2 (countryside) of the Melbourne area, provided the wind flow is not influenced by topographical or other features that may cause an increase in speed.

Preliminary racking tests demonstrated the diaphragm action of the wall cladding and ceiling lining in resisting the horizontal racking forces. Later racking tests showed that the conventional diagonal bracing was irrelevant after the walls had been lined. The racking stiffness of the house was closely monitored as the internal lining was systematically removed from each face of most of the internal walls. Even after removing that lining, the house could still easily resist the horizontal racking forces, albeit with larger deflections.

Lateral pressure tests on the brickwork skin showed that in its uncracked state it was easily able to resist both the positive design pressure and suction design pressure. The brickwork was stronger in resisting positive pressures than negative pressures because of the shape of the wall ties used.

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1. INTRODUCTION

This Melbourne style brick veneer house is the second in a series of three research houses to be built and tested for the Australian Uniform Building Regulations Co-ordinating Council. The first house was also of brick veneer construction, but built for cyclone prone areas of Australia. The performance of that house has been reported elsewhere (Reardon, 1986).

In the Cyclone Structural Testing Station's overall research programme into the performance of housing under simulated wind loading, this Melbourne style house is the fifth new house to be tested. The response of the other houses has been reported by Boughton and Reardon (1983), Reardon and Boughton (1984), Boughton and Reardon (1984) and Reardon (1986).

All of the houses tested to date have been subjected to simulated cyclone winds, but it is well known that other winds can also cause significant damage to housing. Brisbane's annual thunderstorm season at the end of each year is renowned for lifting roofing off older houses and damaging roof structure. Whilst the other capital cities do not have such a pronounced season of risk, they certainly have a significant amount of wind damage to buildings. It was therefore logical to include in the test series a house built for non-cyclone regions and test it for winds of the speed it would experience in practice.

As the Queensland Department of Local Government has in recent years produced and amended its excellent Home Building Code (1986), there appeared to be no valid reason to test a house built according to those provisions. There is little doubt that the Code contains the best set of provisions in Australia for building houses to resist non-cyclone winds. It was eventually decided to build a Melbourne style house as being representative of houses in capital cities.

While it is acknowledged that there will be some differences in construction between the test house and what may be claimed to be the typical Sydney house or the typical Adelaide house, it is believed that the Melbourne test house consists of typical construction details that could be used in any capital city.

2. THE TEST HOUSE

2.1 Design

In order to ensure that the house construction represented that being built in Melbourne, co-operation was sought from Jennings Industries, the large Melbourne building company that specialises in house construction. It had been previously decided to keep the plans of this Melbourne test house identical to those of the brick veneer house that had been tested for cyclone conditions. This would save costs by being able to reuse the same concrete floor slab and by not having to relocate the large loading frames. It would also allow direct comparison of the performance of the two houses. Therefore, based on the plans supplied by the Cyclone Testing Station, Jennings provided a set of documentation for the construction of the house. The documentation included the following:

Ground floor plan and external elevations
Associated wall section detail
Wall framing guide
Ceiling framing guide
Roof framing guide
Eaves layout guide
Complete bill of quantities.

This ensured that the house components would be identical to those of a standard Jennings house. Figure 1 shows the north and south elevations of the test house. For nearly all of the tests the wind was considered to approach from the north, thus the north elevation was the windward face. Figure 2 shows the plan, overall dimensions and approximate room dimensions.

In order to ensure that the house construction would also be of typical Melbourne building practice, two specific decisions were made. All of the timber used for the construction was sent to Townsville from Melbourne and a team of sub-contractors was sent by Jennings to erect the timber frame. Thus the timber framework represented Melbourne building practice in every detail.

It was not considered so essential for the other components of the house to be transported from Melbourne as they were manufactured items. Therefore the roof tiles, bricks and lining materials were supplied from Brisbane. The wall ties were sent from Melbourne as they were considered likely to have an influence on the overall performance of the brickwork. The particular ties used were those specified in the Jennings documentation. Although the roof tiles came from Brisbane, they were

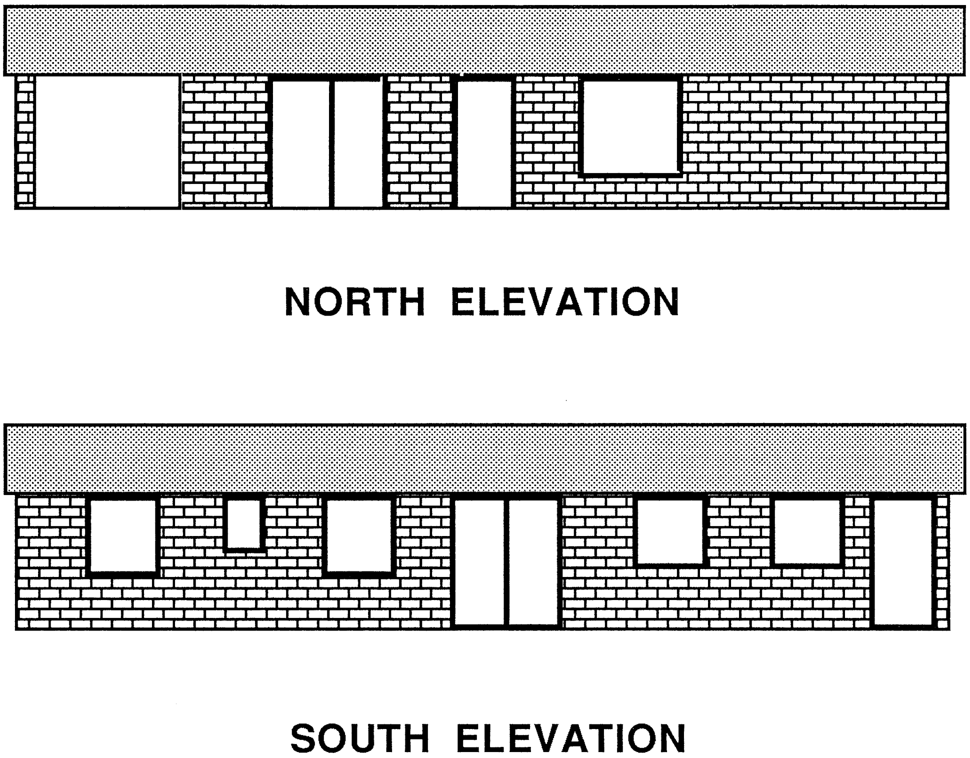


FIGURE 1 Elevations of the test house

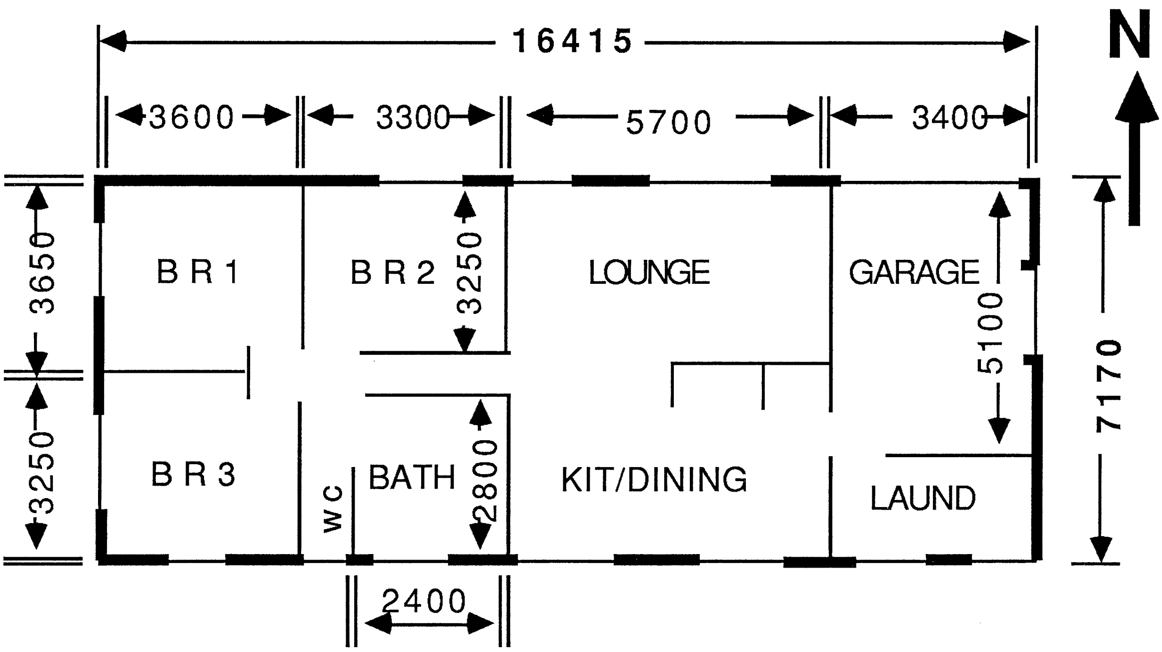


FIGURE 2 House plan with approximate room dimensions

fixed in accordance with the normal practice used in Melbourne. Although the bricklayer lived in Townsville, he had served his apprenticeship and gained his early trade experience in New South Wales.

The test house was therefore considered to be of typical construction for areas not affected by tropical cyclones. From the engineering viewpoint the timber frame was built in accordance with the provisions of the Timber Framing Code, AS 1684 (SAA, 1979). This code specifically states that its recommendations may not be satisfactory for use in areas where the design wind velocity is in excess of 33 m/s. This wind velocity, which represents terrain category 3 in a non-cyclone area, was therefore used as the maximum design velocity for the test house. It should be noted that this design velocity is actually conservative for Melbourne as the code is meant to be used Australia wide.

2.2 Construction

The brick veneer house was of rectangular plan 16.415 m long and 7.17 m wide, with gable ended roof construction. It was built on a concrete slab and had a tiled roof of 18 degree pitch. Only those construction details that were considered to affect the strength of the house were included. Thus there were no windows or doors, no fittings, no cupboards, no plumbing or electrical wiring or any other detail not considered to be structural. Despite the absence of windows and doors, they were considered to be in place for the calculation of horizontal wind forces on the walls. For example, the lintel beam in the garage was loaded during test with the equivalent horizontal reaction that would be generated from wind blowing on a large garage door.

All timber was unseasoned Victorian hardwood of stress grade F8.

A summary of the main construction details is given below.

Floor slab: The slab was 100 mm thick concrete cast over compacted fill. It had edge beams and some internal beams, however as the slab had been designed for a previous test house the internal beams were not necessarily directly below internal walls. There was no step down edge for the brickwork, but this was considered inconsequential.

Wall frames: All wall frames were 2400 mm high. Common wall studs were of 100 x 38 mm cross section spaced nominally 600 mm apart. Studs beside window openings and doors wider than 900 mm were 100 x 50 mm.

Top and bottom wall plates were 100 x 38 mm. All walls had one row of 75 x 38 mm noggings staggered at approximately mid-height. All internal walls and external wall sections of sufficient length were braced diagonally with 50 x 25 mm members let into the studs. Where two walls were aligned the braces sloped in opposite directions.

Lintel beams were 150 x 50 mm spanning either 1800 or 2100 mm.

The frame was constructed with minimal fastening. Two power driven 75 x 3.05 mm smooth shank nails were used to attach plates to studs and noggings to studs. The nails were driven into end grain on both occasions. The bottom plate was fixed to the slab with explosive driven nails at 600 mm spacing, within 100 mm of a stud.

Joints in the top plate were always located over a stud. Where a joint was required, the two parts were joined with a small toothed plate connector. Walls at right angles to each other were joined with two such connectors hammered into the plates.

Ceiling structure: The ceiling joists were 100 x 38 mm at 600 mm spacing, and had a maximum span of about 2800 mm. They were supported by hanging beams of 170 x 38 mm or 200 x 38 mm, depending on the span of the hanging beam. In the ceiling of the lounge the hanging beams were in turn supported by 3200 mm long 195 x 50 mm beams.

Where the ceiling joists were joined at a wall they were lapped about 250 mm, secured together with 3 nails and were skew nailed to the wall with 2 nails.

Where there was no internal wall available, the ceiling joists had a floating lap joint about 1500 mm long fastened with 6 nails. At the external walls the joists were also fastened with 2 skew driven nails. There were no ceiling battens.

Noggings were fitted at 600 mm spacing between ceiling joists to provide lateral support to those walls parallel to the joists. The noggings were skew nailed to the walls.

Roof structure: Figure 3 shows a typical cross section of the pitched roof construction. The 125 x 38 mm rafters were at 600 mm spacing and located directly over studs, except where they were over

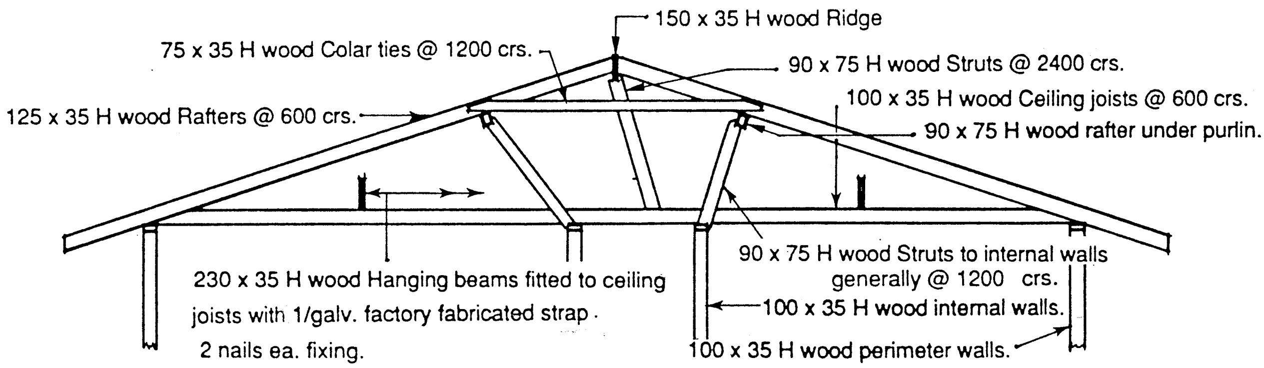


FIGURE 3 Typical Roof Construction

openings. The rafters were attached to the 175 x 38 mm ridge beam with two skew driven nails. They were attached to the external walls with one nail skew driven into the top plate and another driven into the ceiling joist. The rafters were also supported internally by underpurlins which reduced their maximum span to 2500 mm. They had a 600 mm overhang. Collar ties of 75 x 38 mm section were attached to every second pair of rafters with two nails at each end. At the gable end 100 x 38 mm outriggers supported the verge.

The 100 x 75 mm underpurlins were strutted normal to the roof slope by 100 x 75 mm members at 1200 mm spacing. Where the plan would not allow strutting of the underpurlin, it was stiffened with a 195 x 45 mm beam of dry dressed hardwood bolted to the underpurlin at 900 mm spacing. The stiffened underpurlins in the garage spanned 3300 mm while those in the lounge spanned 3000 mm.

The ridge beam was strutted at about 1900 mm spacing by 100 x 75 mm members. At each end of the ridge beam a 100 x 38 mm wind brace, approximately 3200 mm long, extended diagonally back to a suitably located hanging beam or strut. The wind brace was secured at each end with 2 or 3 nails.

Battens and tiles: Roof battens of 50 x 25 mm cross section were spaced approximately 330 mm apart and fastened to the rafters with one 50 x 2.8 mm plain shank flat head nail per crossover. The concrete tiles were laid with every third row nailed to the battens with 60 x 2.8 mm galvanized flat head nails. The order of fixing was to nail the second row from the eaves and then every third row.

Internal linings: Most walls were lined with 10 mm gypsum plasterboard and fixed in accordance with the manufacturer's specification for normal use. Horizontal joints between sheets were finished with jointing cement. A narrow cornice (70 mm around the inside arc) was used at the top of all walls.

In the bathroom, 6 mm fibre cement sheets were fixed to the wall framing according to the manufacturer's specification for normal use. The same finishing cement and cornice were used. In the garage and laundry 4.5 mm fibre cement sheets were installed vertically. They were nailed to the frame, butt jointed at a stud and had no jointing cement.

As the ceiling joists were spaced 600 mm apart, 13 mm plasterboard had to be used for most ceilings. In the laundry and garage 6 mm fibre cement board was used for the ceiling. Timber quad cornice was used in these areas.

Brickwork: Standard metric clay bricks were used for the construction. Each brick had three holes approximately 40 mm diameter to assist the bond to the mortar. The wall ties were as specified by Jennings Industries and were sent from Melbourne. The 100 mm long ties were made from 0.8 mm galvanized steel with one concertina shaped end, about 40 mm long, to provide suitable bond with the mortar. For the rest of its length the tie had a U-shaped cross section. Figure 4 shows a tie. They were face nailed to the timber studs with one 30 x 3.15 mm nail to align with every fifth course of brickwork. This resulted in an effective spacing of 600 x 430 mm. As the wall was 26 brick courses high, the top bed of mortar was tied to the timber frame.

The east wall of the house, which was the garage wall, was of single brick construction and incorporated four 350 mm square piers.

A damp proof course membrane (d.p.c) was used between the bottom course of bricks and the floor slab. As recommended by the Brick Development Research Institute, half a thickness of mortar bed was placed directly onto the slab, the d.p.c. was installed and the other half of the mortar bed was laid. The

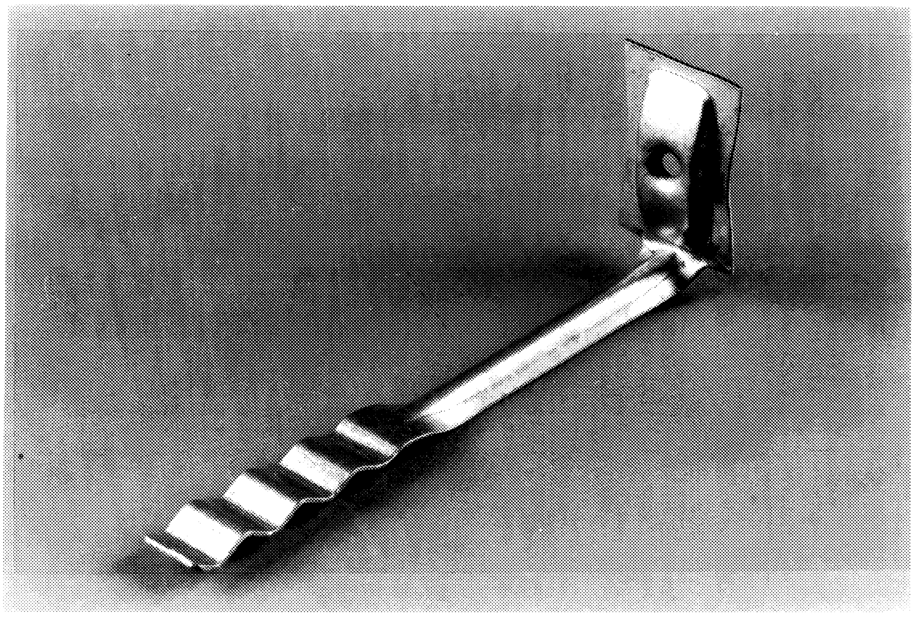


FIGURE 4 Wall Tie

mortar was mixed from local ingredients using a 1 : 4 cement-sand ratio.

A test pier of brickwork, 9 bricks high and 3 wide, was made during each day of construction. Each of the piers was laid by the same bricklayer in the crew.

3. LOAD SIMULATION AND RESPONSE MEASUREMENT

3.1 Determination of Wind Loads

Wind loads on domestic buildings are usually determined from the provisions of the Wind Loading Code, AS 1170 Part 2 (SAA, 1983). They are therefore often conservative when considering a particular design, as codes must be written for the worst likely combinations of events. The alternative method of determining wind loads, by conducting wind tunnel tests, would be far too expensive for individual houses. While this code approach is suitable for the structural design of normal houses, it is not totally acceptable for the case of the research house. In order to accurately determine the loads on the research house, wind tunnel results from a very similar house were studied.

During the time that the house was being tested a draft revision of AS 1170, Part 2 - 1983 was issued. The main reason for the draft was to change the format of the code from a deterministic design approach to a limit states design approach. Design loads for the test house were also calculated from the provisions of the draft and used for comparative purposes. It must be emphasised, however, that the draft document was still out for public comment at the time of testing, and any revisions that may result from that comment could not be considered for the test house.

3.1.1 Using AS 1170, Part 2 - 1983

The Regional Basic Design Wind Velocity for Melbourne at a 10 metre height for a 50 year return period, taken from Table 2 of the code, is 39 m/s. However, although this house has been referred to as the Melbourne house, it is really meant to be typical of construction in the southern regions of Australia. Hence basic wind velocities for Sydney and Brisbane should also be considered. Table 1 lists the Basic Regional Wind Velocity and the design wind velocity for a height up to 5 metres in terrain category 3 (suburban environment) in each of those capital cities.

TABLE 1
DESIGN WIND VELOCITIES
(From AS 1170 Part 2, 1983)

	Design Wind Velocity (m/s)	
Location	Basic	Terrain category 3 (5 m height)
Melbourne	39	25
Sydney	44	29
Brisbane	50	33

When testing a house to destruction, a number of decisions must be made in the planning stages that cannot be revised as the tests progress. One of these is the assumed direction of approach of the wind. Given that the simulation can be made only for the orthogonal directions parallel or perpendicular to the ridge line, consideration must be given to the local and global effects of each. For buildings with very low pitch roofs, the code gives the same maximum uplift pressures (suctions) on the roofing for wind approaching from either direction. But when the wind approaches parallel to the ridge line, only the end portion of the roof experiences maximum pressures, whereas wind perpendicular to the ridge line causes maximum pressures along the complete length of the roof. It is obvious then that wind impacting the building normal to its length would cause greater stresses due to combined uplift and racking than would wind from the other direction.

As the roof on the test house was pitched at 18°, the maximum pressure coefficient on the roof for wind normal to the ridge line was considerably lower than that for wind parallel to the ridge line. Figure 5 shows pressure coefficients from the code for wind normal to the ridge line of the test house. Figure 6 shows the external pressure coefficients for the roof with the wind parallel to the ridge. The coefficient on each slope is the same. The internal pressure coefficient for this case was also taken as +0.2. Although the latter wind direction produces higher local stresses, the global effects are probably less than those for the former case.

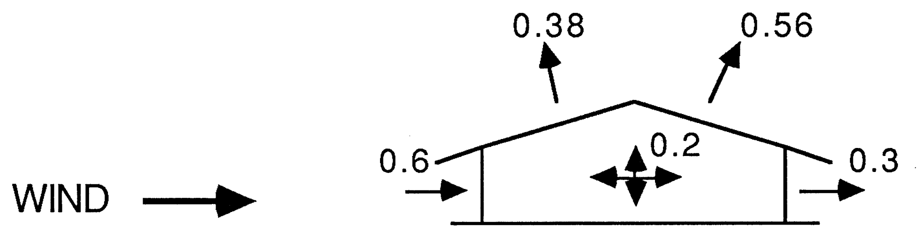


FIGURE 5 Pressure Coefficients from the Code - Wind Normal to Ridge

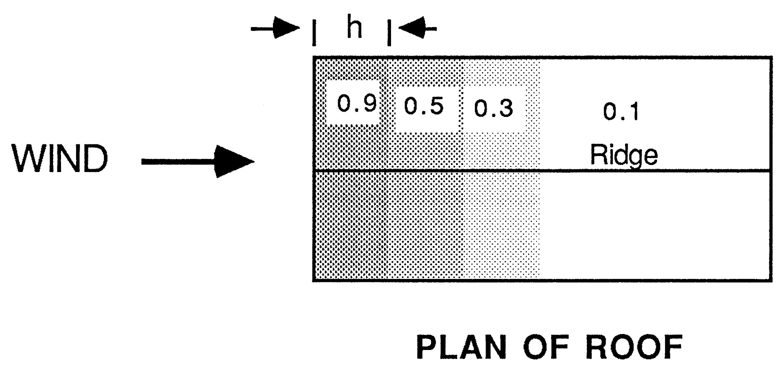


FIGURE 6 External Uplift Pressure Coefficients - Wind Parallel to Ridge

Constraints from the loading equipment also influenced the choice of which wind direction should be considered as the design case. The equipment had been initially designed to simulate forces due to wind acting normal to the ridge line and it would have been very expensive to alter it.

The design case was therefore taken to be with the wind acting normal to the ridge line (Figure 5). However, as the test was to be by static loading the final results could be analysed to give an estimate of the likely performance of the house for the case of wind in the orthogonal direction.

As indicated on Figure 5, the pressure coefficient inside the house was taken as +0.2. This value assumes that the windward and leeward walls are equally permeable and the side walls are impermeable. It could also imply a dominant opening on the windward wall with a permeability of about 1.25 times the sum of permeabilities of the other walls. This value of +0.2 is somewhat arbitrary, but it reflects the Station's opinion that some positive internal pressure should be considered for the design case. The Station considers the use of the maximum internal pressure coefficient of +0.8 to be too severe a design case for non-cyclone areas, where there is little likelihood of flying debris to shatter windows.

3.1.2 Using draft document DR 87163

In terms of the draft code, both Melbourne and Sydney are located in Region A which has a regional basic design gust wind speed (V_p) of 41 m/s for permissible stress design. Brisbane is located in Region B, with a higher V_p of 49 m/s. These basic wind speeds are then modified for terrain, height and shielding, if appropriate. The combination of these multipliers results in a modification factor of 0.66 for a height of 5 m in terrain category 3 provided that the house being designed has shielding from at least two rows of houses upstream. These houses must have an average spacing of five times their height, that is, almost 14 m for the test house. Table 2 lists the regional design velocities.

TABLE 2
DESIGN WIND VELOCITIES
(From draft wind loading code)

Location	Design Wind Velocities (m/s)	
	Basic	Terrain category 3 (5 m height)
Region A (Melbourne, Sydney)	41	27
Region B (Brisbane)	49	32

This draft code introduces more variables than were in the previous editions, with a consequence that it is more open to interpretation. The terrain category 3 wind velocities for Regions A and B could equally be interpreted as 31 and 37 m/s respectively, if the average spacing of the upstream houses was ten times their height rather than five times. Likewise, intermediate values would also be valid. The above values have been chosen in this instance simply because they are more in line with the current code recommendations.

Pressure coefficients relating to the test house are the same in the draft code as they are in the 1983 edition.

3.1.3 Using wind tunnel results

Wind tunnel tests were not made on a model of this test house as the results of a similar house were available. Roy and Walker (1984) measured the total forces on the model of a house 14 x 7 m with a gable roof and presented them in the form of force coefficients, defined as the ratio of the pressure based on the projected wall or roof area to the dynamic pressure based on the mean velocity at eaves height (3 m). The model used by Roy and Walker was effectively sealed, whereas the test house was considered to have two long opposite walls equally permeable and the side walls impermeable. Hence the vertical force coefficients were adjusted using the appropriate internal pressure coefficients given in the Wind Loading Code and assuming that the external and internal pressures were independent of each other. Table 3 lists the total force coefficients for two different wind directions in the case of 18° roof pitch.

TABLE 3
Total Force Coefficients from Wind Tunnel Tests

	Wind normal to ridge line	Wind at 60° to ridge line
Horizontal force coefficient	0.95	1.08
Vertical force coefficient	0.69	0.87

3.1.4 Nominal test design loads

The designation of a design load for a test simulating non-cyclone wind load conditions is not so critical as it is for the simulation of gust wind effects during a tropical cyclone. Simulation of cyclone winds involves the application of a number of load cycles at different percentages of design load, therefore design load must be accurately defined prior to test for the results to be meaningful. For non-cyclone wind effects, static loading can be applied and increased in increments until failure occurs.

Comparison with the calculated design load (or with any number of different design loads) can be made mathematically in retrospect.

Considering pressure coefficients for the walls, all of the available information is in reasonable agreement. Both the current and the draft codes recommend a pressure coefficient of +0.6 on the windward wall and -0.3 on the leeward wall. The algebraic sum of these pressure coefficients, 0.9, agrees reasonably well with the maximum horizontal force coefficient listed in Table 3.

Although the pressure coefficients shown in Figure 5 appear as though they would result in a net uplift force coefficient significantly less than either of the values listed in Table 3, they actually represent a value of 0.66. However, in general, the coefficients from the wind tunnel tests were higher than those recommended by the Wind Loading Code. In determining the nominal test design loads, it was decided to use the coefficients from the Code as they would have formed the basis on which the test house was designed.

It is to be noted that because of the pressures and suctions acting on the eaves overhangs (Figure 5), the two support reactions are nearly equal. Figure 7 shows the calculated state of equilibrium in terms of pressure coefficients for the roofing system, assuming support at the external walls only. The reactions R_A and R_B are expressed as pressure coefficients based on the planform width of the roof.

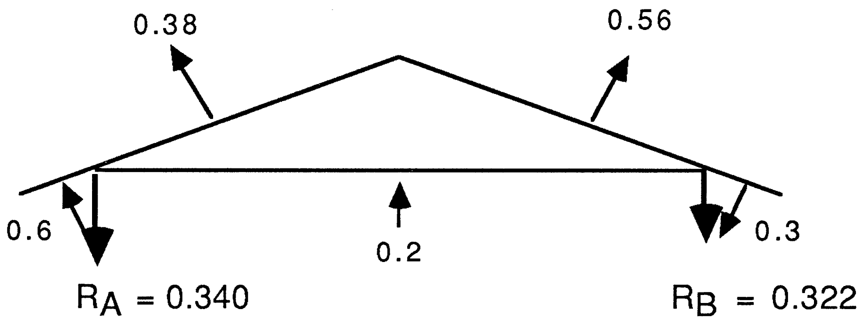


FIGURE 7 Pressure Coefficients and Calculated Reactions

Because the two reactions calculated from the coefficients were similar, it was considered satisfactory for the test loading to apply two equal uniform loads to the roof slopes. The application of equal loads makes the test procedure much simpler. Figure 8 shows the pressure coefficients used to calculate the uniform loading which would be applied to each slope to produce reactions within 3% of those shown in Figure 7. It should be noted that the uplift coefficients are based on the sloping

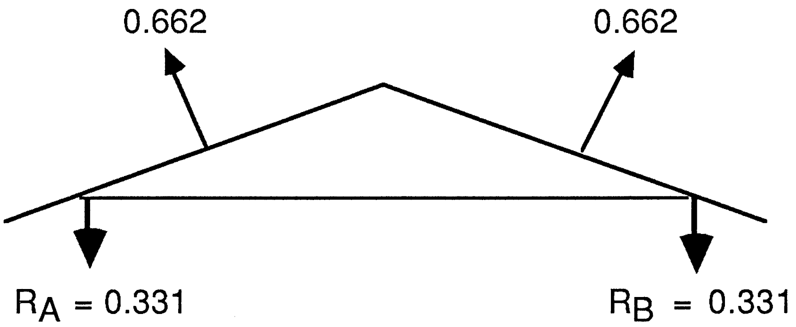


FIGURE 8 Uniform Uplift Test Loading and Reactions
(Expressed as pressure coefficients)

surfaces whereas the reaction coefficients are based on the total width of the roof.

Using the uniform loading configuration shown in Figure 8, test design loads can be calculated for each of the capital cities previously mentioned or for any other location of interest. Table 4 shows the calculated average test design uplift and lateral pressures for category 3 terrain in Melbourne, Sydney and Brisbane as well as for Regions A and B as defined in the new draft code. The pressures are designated as average because they are those considered to be acting over the whole surface of the roof or walls. Higher local pressures, in the order of twice those listed, can act on small cladding elements and their immediate supports. These local pressures were not applied to the test house as it was considered that cladding elements are generally better tested in isolation in the laboratory than as part of the house. However the effect of the high local pressures will be considered when analysing the final results.

TABLE 4
AVERAGE DESIGN PRESSURES FOR TEST

Location	Average design pressures (kPa)	
	Uplift	Lateral
Melbourne	0.25	0.35
Sydney	0.32	0.44
Brisbane	0.42	0.57
Region A	0.28	0.39
Region B	0.40	0.55

3.2 Application of Loads

The loading system consisted of twelve large frames, six spaced evenly along each long side of the house. The forces were applied by means of hydraulic rams pulling on cables which were in turn connected to a load spreading system. All rams were connected to the same hydraulic pump, but controls allowed the pressure in the horizontal rams to be different from that in the vertical rams. This allowed the simultaneous application of independent uplift and racking forces on the house. Every effort was made to ensure that the pressure in each ram of a particular group was the same.

During the test programme four different loading systems were used:

- ◊ Combined uplift and racking
- ◊ Uniform racking
- ◊ Individual racking
- ◊ Uniform lateral pressure loading.

Each loading system involved at least one hydraulic ram pulling on a cable that was attached to part of the house. The ram was usually loaded in increments to a predetermined force measured by an electronic force transducer in series with the cable. At each increment horizontal and vertical displacements of the house were measured by electronic displacement transducers at numerous locations.

The different methods of load application will now be outlined.

3.2.1 Combined uplift and racking

Uplift forces, simulating the combination of uplift pressure on the roof surface and internal pressure acting on the ceiling, were applied to the roof structure by means of the twelve large loading frames, six loading each slope of the roof. Figure 9 illustrates the loading system. The hydraulic rams "a" pull down on one end of the large "see-saw" beams "b" causing uplift forces on load spreaders "c" attached to the roof. Each load spreader distributed the applied force over an area of 12.76 m^2, that is, the 4.4 m length of roof slope multiplied by the 2.9 m spacing of load frames. Each load spreading set reduced the applied load to sixteen equal portions which were then distributed to the underside of the roofing battens. Thus the uniform uplift pressure was simulated by a total of 192 uplift forces distributed evenly over the roof surface. Figure 10 illustrates the load distribution system.

It should be noted that this method of loading does not impose any load directly onto the roof tiles. Although in this instance the decision was

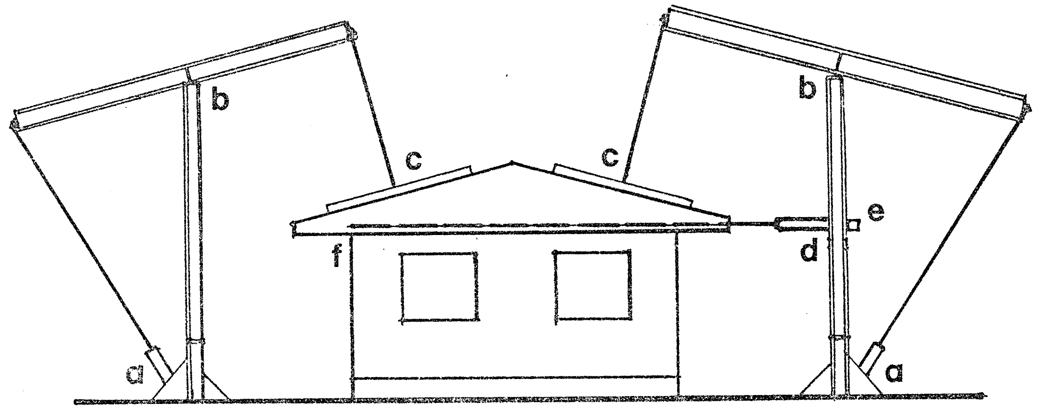


FIGURE 9 Loading System for Combined Uplift and Racking

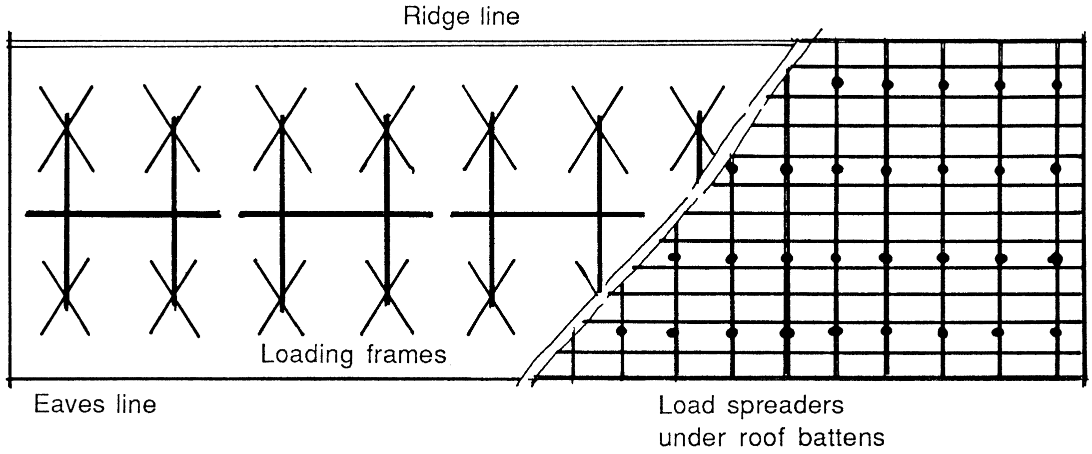
made because of loading constraints, it is accepted that the performance of roof tiles under wind loading can be better assessed in the laboratory.

Figure 9 also illustrates the system used to apply the horizontal racking forces. Four horizontally mounted rams "d" were attached to a large RHS steel beam "e" fixed to the uplift loading frames at wall height. A cable was extended beneath the ceiling from the ram to a load spreading system "f" at top plate level on the windward wall. Each ram load was distributed to four loading points approximately one metre apart. Thus there were sixteen loading points simulating the uniform racking force along the top plate.

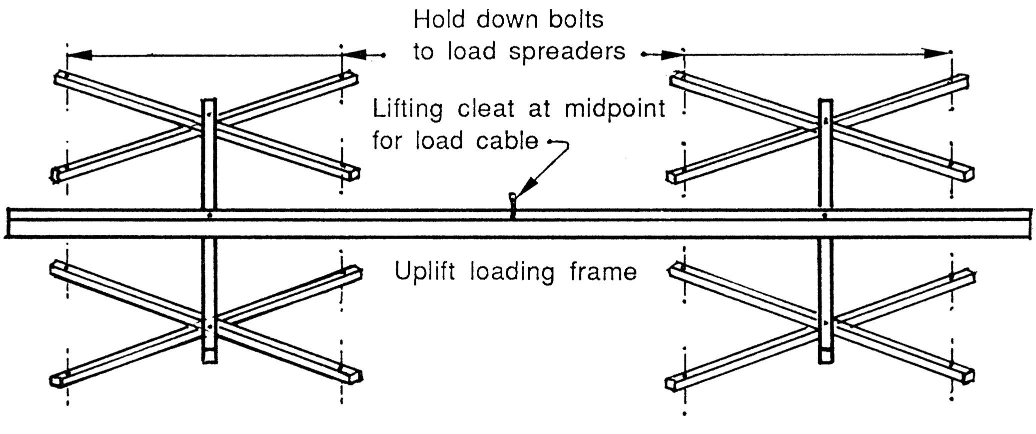
The racking force applied to the top plate was calculated as half of that caused by the sum of the pressure on the windward wall and the suction on the leeward wall, plus the total horizontal component of pressures on the roof slopes. When incrementing the forces care was taken to maintain the correct ratio between uplift and horizontal pressures.

3.2.2 Uniform racking

When only the uniform racking forces were applied to the structural framework of the house, the method of application was exactly the same as for combined uplift and racking. That is, the uniform loading was simulated by sixteen concentrated horizontal loads approximately one metre apart along the top plate of the windward wall. This method of loading was used for some tests during construction of the house and during investigation of the bracing performance.



(a) Loading Arrangement



(b) Load Spreaders

FIGURE 10 Details of Uplift Loading System

3.2.3 Racking individual walls

During construction of the house, each transverse wall was racked a number of times to determine its response to the addition of potential diaphragms, namely the roofing and the ceiling. Details of the measured response are given in Section 4.

To rack a wall, a ram was aligned slightly to one side of the wall and a cable was attached to the top plate of the windward wall as near as practicable to the wall being racked. This usually resulted in the line of action of the racking force being about 50 mm from the face of the wall.

The same loading system was used to rack individual walls during the final tests to destruction. However when racking the brickwork, where no convenient top plate was available, a cable was used along each face of the wall with a bridging piece applying the load to the top course of brickwork. This system was also used for some walls which had a high racking strength.

3.2.4 Uniform lateral pressure loading

Although the brick skin is not considered to be a structural element in a brick veneer house, it still must have the structural capacity to transfer horizontal wind pressures to the timber frame. This is accomplished per medium of the wall ties which act as a link between the brickwork skin and the framework. These ties may have to act in tension when the brickwork is subject to suction pressures or in compression when positive pressure is acting.

A decision had to be made as to the appropriate direction of loading the brickwork, using either positive pressure or negative pressure (suction). As the pressure coefficients for each direction are the same, the design did not dictate which direction should be tested. The wall ties were quite different from those used in the previous brick veneer test house and therefore those results offered little information about the likely performance of this house. It was decided therefore to conduct positive pressure tests on the designated windward wall, and some suction tests on parts of the leeward wall. There was also the possibility that some parts of the windward wall may not be damaged too much and could be retested under suction loading.

Figure 11 illustrates the loading device used to apply uniform positive pressure to the brick skin. The stiff frame was hinged to the edge of the floor slab and had a cable attached to its top centre. Five such frames, each approximately 2.5 m wide, were used to load the brickwork along the front of the house. Large air bags were inflated between each hinged frame and the brickwork. The bags provided a cushion between the loading frame and the brickwork and ensured that the test pressure was applied evenly. The predetermined uniform pressure was applied by pulling on the cable with a calculated force, using a hydraulic ram, and squeezing the passive air bag against the brickwork. Where there was a window opening an infill panel was inserted to simulate the effect of the window and transfer the applied force to the timber frame. This system was not used, however, for the doorways in the lounge room area.

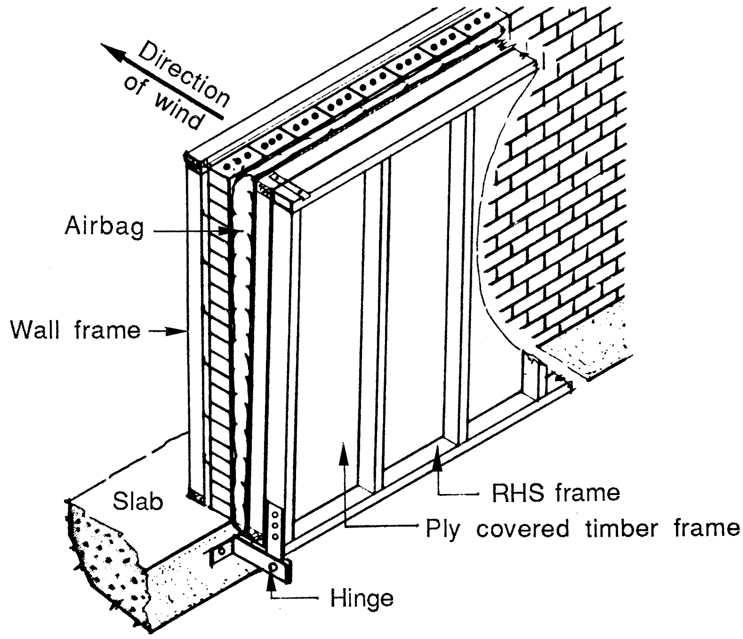


FIGURE 11 Loading Frame for Applying Positive Pressure to Brickwork

The test pressure used for the brickwork was calculated on the basis of wind on the external windward face only. It was not appropriate to apply the sum of windward and leeward pressures as was used for the structural wall framework.

3.3 Constraints on Loading

In most simulated loading programmes, compromises have to be made to accommodate the constraints imposed by the loading system or its ancillary equipment. Such compromises must be kept to a minimum, but if they are unavoidable an accurate assessment of their likely effects should be made.

Although it has just been demonstrated in Section 3.2.4 how uniform pressure can be applied to walls, the application of uniform suction is more difficult. The method used in this series of tests was to apply a series of small point loads. This method is an acceptable alternative which increases in accuracy with the number of load points.

When the large uplift loading frames were set out for the combined uplift and racking tests, extreme care was taken to ensure that they were spaced uniformly along the length of the house as well as being located at the appropriate position to ensure that the uplift cables were pulling perpendicular to the roof slopes. While this method of loading provided the equivalent of a uniform uplift pressure on the roof surface, its net effect was to apply along each slope 24 lines of loading from eaves to ridge. Figure 10 (a) illustrates this point.. As the rafters were spaced 600 mm apart and the lines of loading were at 725 mm spacing, the loads applied to battens were sometimes very close to the rafters. The worst case was at rafter number 11 (numbered from the west) where the line load was only about 40 mm from the rafter and thus this rafter may have received a greater load than the average and the rafter next to it would have received a smaller load. However the loading system left no rafter unloaded and only four had the line load within 100 mm, compared with the maximum possible distance of approximately 360 mm.

3.4 Response Measurement

In order to interpret the behaviour of the house, accurate measurement of its response had to be made and recorded. As has already been stated, applied loads were monitored by electronic force transducers linked in series with some of the cables used to apply the loads. Displacements were measured by electronic displacement transducers and fed to a micro computer for processing and storage. The data acquisition system, which was specifically developed for the house testing research programme, has been described elsewhere (Boughton, 1983)

Displacement measurements were made at up to 70 locations on the house, depending upon the test being conducted. For the combined uplift and racking tests, vertical displacements at the ends of the rafters and of the adjacent top plates were measured as well as horizontal movement of the top plates. Horizontal and vertical displacements were also measured at the bottom plate level for each transverse wall. All displacements were measured relative to ground via sets of independent scaffolding. The transducers were fixed to this datum by stands with magnetic bases, which allowed easy portability from one location to another if necessary. Figure 12 shows typical locations for measuring displacements of the roof and wall.

The load and deflection data were stored by the micro computer during the test and transferred to magnetic tape on completion of the test. During the course of the test, the deflection at any point could be plotted against applied load and from that graph a determination made as to the likelihood of yielding of structural components.

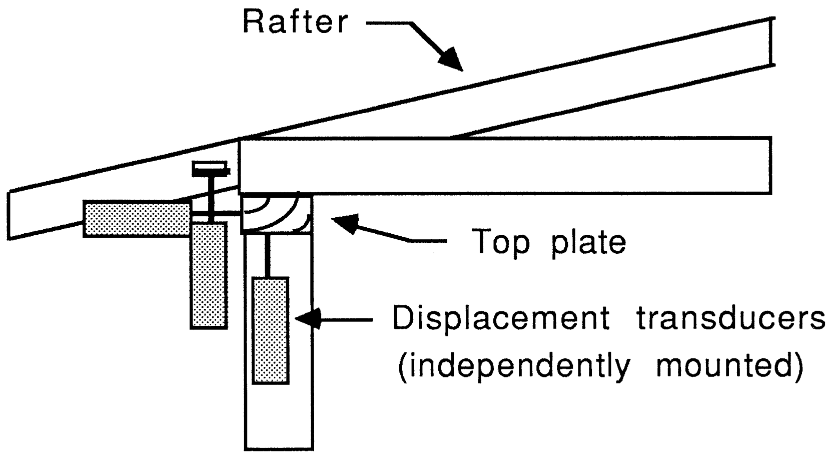


FIGURE 12 Typical Locations for Measuring Displacements

4. NON-DESTRUCTIVE TESTING

During various stages in the construction of the house, each of the transverse walls was racked slightly to determine its change in response as extra cladding was installed. The racking force was applied as a concentrated load to the top plate of the windward wall, as close as practicable to the axis of the transverse wall. Later a uniform racking force was applied as a line load to the top plate of the windward wall. This latter test was to investigate the response of the house between transverse walls, such as at window heads.

For convenience the walls were numbered from the western end of the house, with the external wall being number 1. Each portion of internal wall had a diagonal timber brace, except that length dividing the kitchen and laundry. Thus walls 1, 2 and 3 had two braces and wall 4 had one brace. On the basis of these braces, using the Queensland Home Building Code recommendations, the first three walls could have had a bracing strength of 4.8 kN and wall 4 could have had 2.4 kN. Because of the lack of specified tie down and attachment to the roof structure, the walls of this Melbourne house do not satisfy the Queensland criteria for bracing walls. However, tests on a previous research house (Reardon, 1986) indicate that the walls may well act as bracing walls without the tie down and attachment details. The above values were therefore taken as the best estimate of the design racking strength of the walls.

The first series of racking tests was conducted after the timber frame had been erected, the roof framing installed, all braces fitted and most internal lining attached. The plasterboard was purposely not installed on the windward wall, so that the response of the brickwork could easily be measured during a later investigation. It was assumed that the lack of internal lining on the windward face would not affect the response of the house to lateral loading. At the completion of the first series of tests, the roof battens were fixed and the concrete roof tiles laid. The second series of racking tests was then conducted. The ceiling was then fixed directly to the ceiling joists prior to the commencement of the third series of racking tests. A final series was then conducted after the cornice was fitted.

Figure 13 shows the numbered walls as well as the location of important gauges measuring horizontal displacement for the tests in this non-destructive series. The gauge locations are circled. The north wall was considered to be the windward wall.

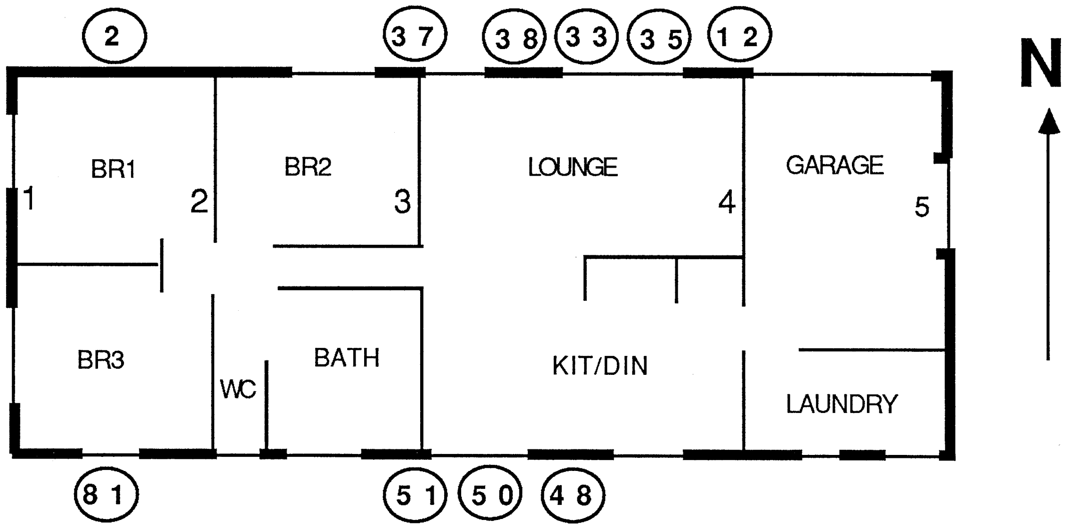


FIGURE 13 Location of Important Displacement Gauges for Non-destructive Racking Tests

None of the brickwork was constructed until after these tests, so there was no attempt made to determine interaction between frame and veneer, or stiffness response of the end garage wall. This also meant that the uniform load applied to the windward top plate could only extend for three quarters of the length of the house, from wall 1 to wall 4. Only temporary props supported the roof structure where the garage brick wall was to be built.

4.1 Racking of Lined Wall Frames

For these tests the entire timber framework and most of the roof structure was complete, the walls had been lined internally but the roofing battens, the roofing and the ceiling had not been installed. Wall 1, the external western wall, had diagonal braces and plasterboard on one face only, whereas each of the other walls had braces as well as lining on both faces. As each wall was racked, it was soon established that the diagonal brace played little part in the bracing strength of the wall. In order to obtain sufficient racking displacement, forces well in excess of the assumed design load had to be used. Wall 1 was loaded to 8 kN and the others were taken to 10 or 12 kN. Each wall was loaded twice to investigate any effect of "settling-in", that is, the taking up of any slack or tolerances induced in construction. A settling-in component was obvious in the measured displacements.

As would be expected, the maximum racking deflection of the walls was quite small, in the order of 3 mm at 10 kN. Table 5 lists the measured racking stiffness of each wall after settling-in. Sometimes the top plate through which the load was transferred moved more than the wall, but it was taking up building tolerances. There was usually very little movement at any other location except at the wall being racked. This is quite understandable as the roof framing had virtually no means of transferring any forces sideways.

TABLE 5
RACKING STIFFNESS OF ISOLATED WALLS

Wall Number	Racking Stiffness (kN/mm)
1	3.2
2	3.5
3	3.6
4	2.6

The uniform racking load produced displacement at many more locations than did the individual wall racking loads. However, the total load applied to the system still had to be in excess of design load in order to produce sufficient deflections. The total load applied to the top plate was 0.86 kN per metre length of top plate, almost twice the Melbourne design racking load of 0.45 kN/m. A notable aspect of this uniform load test was that it produced significant deflections of the leeward wall in between transverse walls. This demonstrated that the pitched roof was able to transfer forces from the windward wall to the leeward one in a compression mode, but was hardly able to transfer forces sideways to the stiff transverse walls. Tables 6 and 7 include displacements measured during the tests.

4.2 Racking After the Addition of Tile Roofing and Ceiling

Prior to the commencement of the first test in this series, the roofing battens were fixed to the rafters and concrete roof tiles were laid by the manufacturer's tradesmen in accordance with the recommendations for roofs in Victoria. Each of the transverse walls was racked in turn and then a uniform racking load was applied to the top plate. All of the displacements were less than those for the frame without roofing. Tables 6 and 7 list the significant displacements measured during each test.

The ceiling lining was fixed to the underside of the ceiling joists according to the manufacturer's specifications. Each wall was again racked in turn and then the whole house racked. There was a significant change in response of the house, as the ceiling acted as a stiff diaphragm to transfer the applied racking forces to other parts of the house. Finally the cornice was installed and each part of the house was retested.

TABLE 6
RACKING DISPLACEMENTS FOR WALL 3 LOADED TO 12 kN

House Configuration	Displacements (mm) Measured at Gauges			
	37	38	50	51
Walls lined	3.2	1.4	1.1	1.2
+ roofing	1.2	1.4	0.4	0.4
+ ceiling	0.8	0.5	0.3	0.4
+ cornice	0.5	0.1	0.0	0.0

The effects of the addition of each of the components is well illustrated in Table 6 which includes the relevant deflections measured as wall 3 was loaded to 12 kN in racking. The location of the gauges is shown in Figure 13. Each line of results in the table is taken from the second of two duplicate tests to eliminate any effects of "settling-in". The last two tests shown in the table were loaded to 15 kN and 18 kN respectively, to obtain reasonable displacement measurements, but the values listed in the table are for the 12 kN load increment.

The most surprising aspect shown in Table 6 is the difference in response due to the addition of the roofing. It is difficult to believe that the discrete roofing tiles were able to act as a diaphragm and thus distribute the applied forces to the other walls. The only obvious answer for the response is the weight of the tiles causing friction between the roof members and the wall top plate.

TABLE 7
LATERAL DISPLACEMENTS UNDER UNIFORM RACKING LOADS

House Configuration	Load Level (kN/m)	Displacements (mm) Measured at Gauges 2	38	33	35	48	50	81
Walls lined	0.86	2.9	1.5	1.5	0.8	1.9	1.7	1.1
+ roofing	0.86	1.0	0.5	0.4	0.4	0.6	0.3	0.1
	1.72	2.1	2.2	2.5	1.4	2.3	2.1	1.3
+ ceiling	1.72	1.0	0.6	0.4	0.4	0.8	0.6	0.5
+ cornice	1.72	0.8	0.2	0.0	0.4	0.3	0.0	0.4
	2.94	1.9	1.0	0.6	0.9	1.0	0.4	0.8

Under uniform racking loading, the windward wall showed a complete change in response as the roofing and ceiling were added. The apparent stiffness of that wall between transverse walls changed significantly as each element was added. Table 7 lists displacements measured at some of the locations shown in Figure 13. It must be emphasised that in order to provide reasonable values for comparison, the displacements have had to be given at different load levels, all of which were well above the design level of 0.45 kN/m for the Melbourne house. If the loading of 0.86 kN/m had been maintained throughout the whole table the displacements for the completed house would have been so small as to be meaningless. However to facilitate the comparison between cladding configurations, the response at two different load levels has sometimes been given.

On first glance the table does not show the large difference in performance caused by the cladding elements. However an analysis of the results of gauge 2 indicates that for the test on the lined walls only the gauge was moving at a rate of 3.4 mm per kN/m of applied force. The addition of the roofing caused this movement to decrease to about 1.2 mm/kN/m. The addition of the ceiling reduced the movement to 0.6 mm/kN/m and the cornice further reduced it to about 0.5 mm/kN/m. That is, the rate of change of lateral displacement was reduced to about 15% of the original rate. While the amount of reduction varies for each of the positions monitored, all of the gauges registered significant reduction in displacement.

5. LOADING OF BRICKWORK

The brickwork caused some problems!

These problems were related to the quality of workmanship, the lack of experience of the Station's staff in visually assessing brickwork quality and the opportunity for people to see the internal face of brick veneer construction. The bricks were laid by a tradesman with many years experience based on a formal apprenticeship in New South Wales. (He advised that some states do not require apprenticeship training for bricklaying.) The authors believe that the bricklayer did his job in the usual manner. He was most obliging and was interested in the Station's research. However the fact that the internal lining had not been fixed on the windward wall allowed the inner face of the brickwork to be inspected. This unusual situation revealed that the brickwork did not conform with some requirements of the Brickwork Code, AS 1640 (SAA, 1974). In Section 5.5, entitled "Workmanship", Clause 5.5.1 states in part that bricks must be laid on a full bed of mortar. In many instances the mortar beds in the test house did not fill the joint between rows of bricks at their inside face. It appeared that the trowel had been wiped on the inside edge of the bricks causing the mortar to slope away from the brick above. A 20 mm wide, 0.5 mm thick ruler was used as a probe to gauge the depth of the voids in various bed joints in the short length of brickwork at the north end of wall 4. Figure 14 illustrates to scale the plan of the mortar on one of the worst bed joints. Lack of full contact is indicated by the dark area. The perpend joints generally had less contact area than the bed joints. On more than one occasion the void in a perpend measured up to 85 mm in depth.

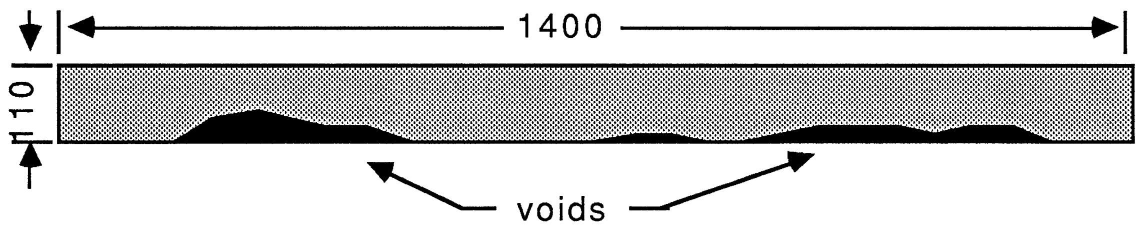


FIGURE 14 Plan of a Mortar Bed Indicating Lack of Contact

When the situation was explained to the bricklayer, he advised that this was normal practice as tradesmen were also required to ensure that there were no mortar droppings at the bottom of the cavity and this method of laying bricks was the best way of ensuring that. Indeed, the bottom of the cavity was very clean.

The Station's staff were then left with the dilemma of deciding whether or not to accept the brickwork. It was clearly not to code requirements but may well represent the norm, as this inner face of brickwork is very rarely available for inspection. Help was sought from others with wider practical experience. Two engineers, an architect, a brick industry representative and a construction supervisor all suggested that the brickwork should be replaced. Their recommendations were accepted.

As it was suspected that the lateral loading on the brickwork would cause failure at one or both of the free standing lengths of the windward wall at the lounge room (see Figures 1 and 2), the bricklayer was asked to rebuild only those short lengths. Because of other commitments he was unable to commence the work immediately. In fact, because of the time delay in assessing the situation the rejected lengths of wall were just 28 days old when they were due to be replaced. This provided the opportunity for them to be tested in their original condition and form the basis of comparison for later tests on the brickwork conforming to the code.

The bricks were of standard metric dimensions, 230 x 110 x 76 mm, were "St Helena" style and had three 42 mm diameter holes through their depth. The mortar was a 1 : 4 cement-sand mixture without any lime or admixtures.

5.1 Tests on Standard Brick Piers

As part of his job the bricklayer was required to lay standard brick piers for mortar joint tests. One pier was laid at the same time each day. The piers, nine bricks high and three wide, were stack bonded and had the perpends carefully broken before the mortar was fully cured. The piers so formed were tested for mortar bond strength after seven days, using the bond wrench technique. The results of all the tests are listed in Appendix A, while a summary is given in Table 8. As can be seen, although the average strength is quite reasonable there was a wide variation in strength. In fact some of the joints had virtually no strength. This was attributed to inexperience in handling when transporting them from building site to laboratory. The joints with virtually no strength have not been included in the statistical analysis.

TABLE 8
SUMMARY OF BOND STRENGTH IN BENDING

Average bond strength	0.328 MPa
Coefficient of Variation	30%

5.2 Lateral Positive Pressure Loading of Brickwork

5.2.1 Original brickwork

Details of the test rig devised to apply a uniform lateral positive pressure to the brickwork have already been given in Section 3.2.4. It has also been stated that the most appropriate loading was considered to be equivalent to a pressure on the windward face pushing the brickwork towards the timber frame. Average lateral design pressures for the brickwork of the windward wall can therefore be calculated for the five likely design cases. The brickwork is considered to be protected from the influence of internal pressures and its calculated average design pressures are listed in Table 9. As can be seen from that table, the pressures on the veneer skin are quite low.

TABLE 9
DESIGN PRESSURES ON THE BRICKWORK OF THE WINDWARD WALL

Location	Average Positive Design Pressures (kPa)
Melbourne	0.23
Sydney	0.29
Brisbane	0.38
Region A	0.26
Region B	0.37

It was considered unnecessary to apply vertical loading to the roof structure during the test on the brickwork skin. Such vertical loading would be resisted directly by the timber frame and its effect on the interaction between the veneer and the framework was considered to be of a secondary order.

The first of the two isolated walls to be tested was the one nearer wall 3. It was 1300 mm (5.5 bricks) long and 2250 mm (26 bricks) high and was attached to four studs with ties at every fifth mortar course numbered from the floor.

During the test the load was increased steadily and the response of both the brick skin and the timber framework were closely monitored. It was soon apparent that even this wall with its incorrectly laid bricks had strength well in excess of the design pressures listed in Table 9. The first obvious failure occurred at a pressure of 2.3 kPa when cracking of the bottom mortar joint was observed. By 2.7 kPa cracking was obvious at the 8th and 16th mortar courses above the floor. However the wall managed to resist 7.6 kPa before it finally broke at the 8th mortar joint and the adjacent studs twisted severely. A detailed inspection showed that the timber studs had suffered a permanent torsional distortion in accepting the load transferred from the brickwork, the ties had bent at the weak corner position where their cross section was flat (see Figure 4). Although the cracking of the brickwork occurred first, it was possible for either failure of the ties at mid height or the bending failure of the brickwork a the mortar joint to have caused the sudden failure of the wall. As the failure was very sudden it was not obvious which of the two mechanisms initiated the failure. Also the stud/top plate joints showed permanent deformation, with one stud having moved about 5 mm relative to the plate. It should be noted that the absence of internal lining and cornice on this windward wall may have contributed to this stud/plate deformation. It might not even have happened had the cornice been in position.

The deflection pattern of this isolated wall gives a good insight into its structural response. For lateral pressures up to 1 kPa, i.e. about four times the design pressure for Melbourne or 2.5 times that for Brisbane, there was practically no movement of either the brickwork or the frame. After this load both of those elements responded in an almost linear elastic manner until failure. It was quite obvious that the brickwork was transferring its loads directly to the frame during that response. That is, the wall ties were able to act very effectively as columns. The displacements of the bricks and adjacent studs nearly matched each other, whereas the bricks between studs moved a little more than the average stud movement. The overall profile of the brick wall, measured between the central studs, showed that at a pressure of 3.95 kPa there was practically no movement of the bottom course, 12.8 mm movement at mid height, but only 3.7 mm at the top course. Figure 15 shows a graph of lateral pressure vs. lateral displacement at mid height of the brickwork between the two studs and the average mid height displacement of the two adjacent studs for pressures up to 3.95 kPa.

On completion of this test it was very obvious that the paucity of mortar in some joints would have had very little effect on the strength of the brickwork when it was being pushed towards the framework, as there was sufficient mortar to hold the ties while they were acting in compression. Time constraints did not allow the fabrication of equipment to conduct a simulated suction test on any of the other walls before the two panels were rebuilt. However the decision was made to conduct suction tests on some of the other walls of questionable construction after the main test programme had been completed.

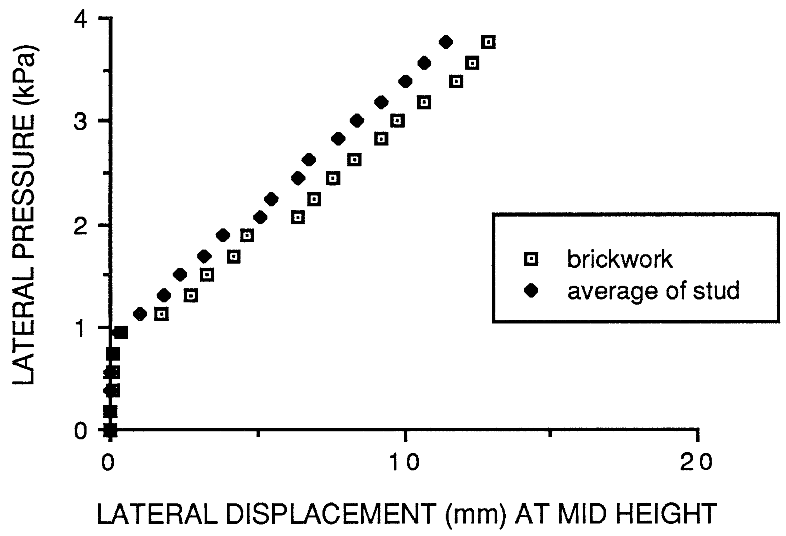


FIGURE 15 Response of First Isolated Wall to Positive Pressure

The second isolated wall, next to the garage, was 1550 mm (6.5 bricks) long and 2250 mm (26 bricks) high. It was also connected to four studs by ties at every fifth course measured from the floor. Once again the uniform pressure was increased in steps and the response of both the brick skin and timber frame was monitored. The first crack was noticed at the 12th mortar joint at 2.9 kPa. The next occurred at the 16th joint at 4.2 kPa and final failure occurred at that same joint at 5.3 kPa. In this test the failure of the wall was caused by a classical bending failure along the mortar joint. The only ties that bent were at joint 11, but this was probably subsequent to the main failure. There was no torsional failure of the studs nor was there any significant permanent deformation between plates and studs.

The lateral response of this wall panel was similar to the previous one until the failure at 4.2 kPa when the brickwork showed a significant movement. As would be expected, the frame also responded at that load level but the magnitude was much less because it was responding to load transfer from the brickwork rather than its own failure. Maximum lateral displacements of up to 27 mm were recorded at mid height just before failure, but these were only about 13.5 mm before the initial failure at 4.2 kPa. Figure 16 shows a graph of lateral pressure vs. the mid height displacement at the centre of the brickwork and the average displacement of the adjacent studs.

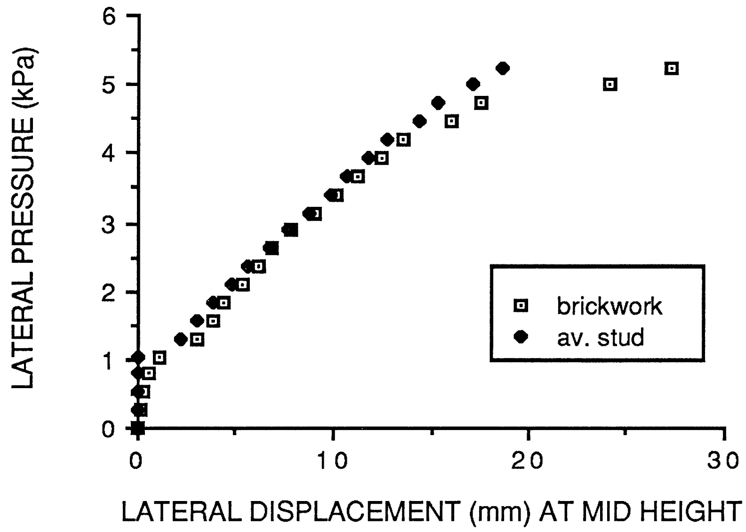


FIGURE 16 Response of Second Isolated Wall to Positive Pressure

5.2.2 After rebuilding isolated walls

Sequentially, these tests were conducted before the combined uplift and racking load tests on the whole house, as it was considered that failure of the brickwork on the windward wall would not in any way influence the performance of the timber framework. This sequence also allowed the plasterboard lining to be left off the windward wall frame so that the inside face of the brickwork could be monitored and observed.

In this test the entire length of brickwork on the windward wall was subjected to positive lateral pressure loading, that is, the brickwork was being pushed towards the timber frame. Five frames of the type shown in Figure 11 were used. The length of wall included the two short lengths of new brickwork of the lounge wall and the original brickwork of the bedroom walls. The window opening in bedroom 2 was framed in and supported by the timber framework, to simulate the effect of wind forces being transferred from the window frame to the studs at the side of the opening. This simulation was not made at the openings in the lounge.

Once again uniform pressure was applied to the walls in increments, with displacements of both the brickwork and the framing being closely monitored. The windowless wall of bedroom 1 was monitored to determine relative movements of brickwork and studs as well as to investigate any difference in its response near the end brick wall.

As was anticipated from the previous tests in this series, the brickwork was very stiff and strong. In fact at the design pressure for Melbourne the maximum displacement was only 0.3 mm. As the pressure was increased gradually most of the gauges recorded a linear response. At eight times the Melbourne design pressure (1.84 kPa) the maximum displacement was only 4.5 mm at about mid height on the brickwork of the isolated wall at the front door. However at ten times the Melbourne design pressure (2.3 kPa), the stud at the side of the window in bedroom 2 suddenly moved 10 mm to a total of 15 mm at mid height. A nearby stud had an overall movement of 10 mm. These movements were a combination of deflection of the stud due to the simulated window loading and a nail slip failure at the joint between the stud and the top plate, but predominantly the latter. The maximum displacement of the brickwork was 9.2 mm at the top of the wall near the internal wall dividing bedrooms 1 and 2. There were other displacements of similar magnitude in the near vicinity. At this stage the test was terminated, leaving undamaged the new brickwork walls in the lounge. The maximum displacement of those walls was only 6.0 mm.

The decision to terminate the test was made on the basis that a load factor of 10 was sufficient to prove any point about the relative strength of the brickwork loaded under positive pressure. Termination of these tests before any catastrophic failure occurred would allow future tests to be conducted on the new brickwork walls, whereby they could be loaded with simulated negative pressures pulling the brick skin away from the framework.

5.3 Negative Pressure Loading of Brickwork

These destructive tests on the brickwork were conducted towards the end of the overall test programme on the house, but the results will be reported in this section. It includes simulated suction tests on both the rebuilt and the original brickwork.

The suction forces acting on the brickwork were simulated by glueing individual loading pads at uniform spacing over an area of brickwork and pulling on them with a hydraulic ram via a wiffle tree. Figure 17 illustrates the principle. Constraints on the loading system were such that only short panels of wall could easily be tested. The system was ideal for testing the short lengths of new brickwork in the lounge and it was tried on some other lengths between window openings. As the strength of the panels loaded in this manner would be very much dependent upon the wall ties, care was taken to ensure that the load was applied symmetrically with respect to the ties. Also, in some instances, the extra ties that are required alongside an opening were cut so that those left for test would be at approximately normal spacing.

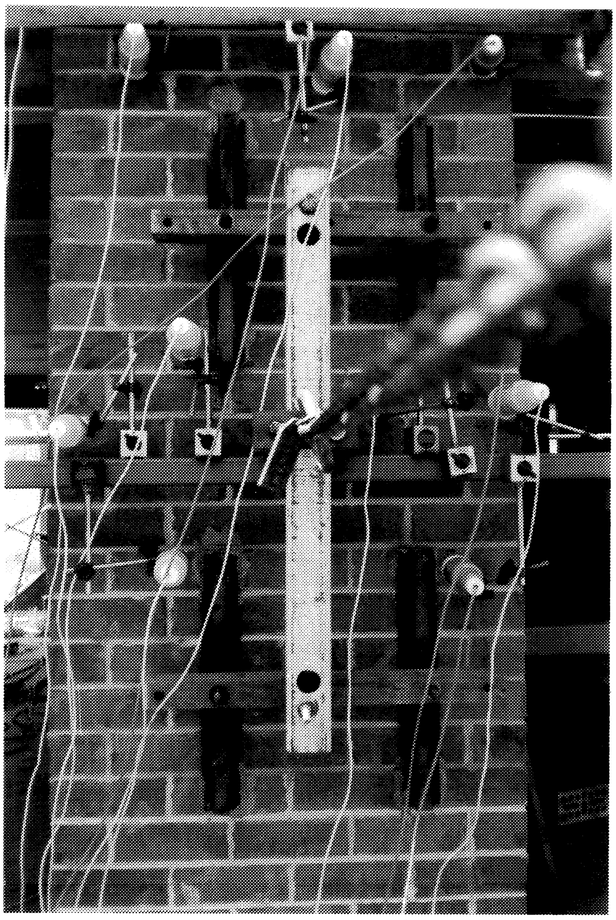


FIGURE 17 Simulated Suction on Brickwork

5.3.1 Rebuilt wall panels

The first test was on the Wall Panel A, the location of which is given in Figure 20. (This was also isolated wall 1 as mentioned in Section 5.2.) It was 1300 mm (5.5 bricks) long and 2150 mm (25 bricks) high. The design pressures listed in Table 9 for wind pushing on the walls are also appropriate for design against average suction pressures, considering the wall being tested to be a side wall relative to the wind flow. The maximum deflection was at mid height throughout the test, that is, the response was a bending mode rather than an overturning one. The failure mechanism was in keeping with this response as the ties at mid height failed before those at the top. The response of the wall was reasonably linear up to 1.1 kPa when the maximum deflection was 6.5 mm at mid height of the brickwork. Figure 18 shows a graph of lateral pressure vs mid height displacement for the centre of the brickwork and an adjacent stud.

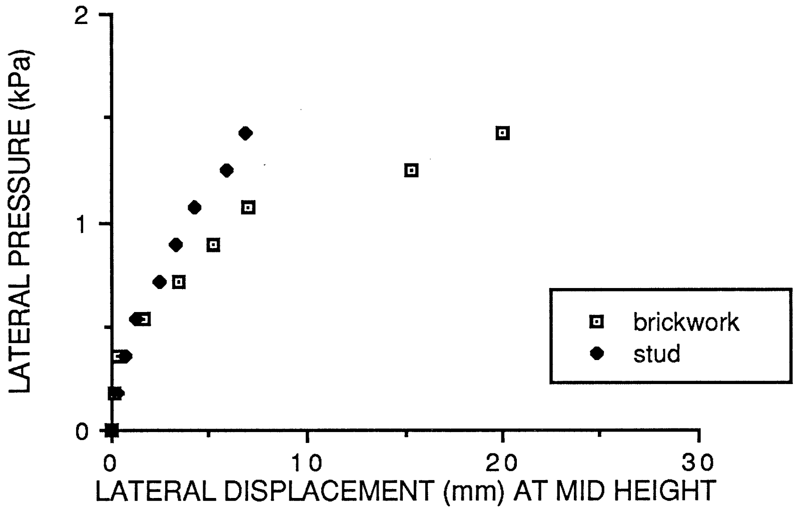


FIGURE 18 Response Wall Panel A to Suction Pressure

The response of this wall to simulated suction forces, as shown in Figure 18, is quite different from its response to lateral positive pressures shown in Figure 15. That figure shows how the brickwork and the studs moved similar amounts at the mid height as the forces were transferred from the brick skin to the framework by the wall ties acting very efficiently in compression. In contrast, Figure 18 shows quite a difference in mid height lateral displacement between the brick skin and the framework. This difference in displacement is due to the relative flexibility of the ties in tension. Under simulated suction pressures two distinct actions were noticeable. The tension force on the tie (see Figure 4) caused the right angle corner to open due to the eccentricity between the acting and resisting forces. In addition, the single nail started to withdraw from the stud. Both of these actions allowed relative movement between the stud and the brickwork. Neither of the actions was able to occur under compressive loading of the tie.

The wall panel eventually failed at a pressure of approximately 1.7 kPa when the brickwork broke in bending at a mortar joint, pulling some ties away from the framing. Figure 19 shows the mode of failure. Although this pressure at failure was sufficiently in excess of the design value for Melbourne to be considered satisfactory, it was considerably lower than the pressure at which the panel had failed when loaded in positive pressure as described in Section 5.2, when the construction of the brickwork was not up to the code specifications. Although the panel had been loaded with positive pressure as part of the test on the whole windward wall there was no evidence of it having been over stressed.

FIGURE 19 Bending Failure of Brickwork

Any overstress would have shown up on the graph in Figure 19. By comparing both the strength and the response of this panel tested for suction forces and its previous version tested for positive lateral forces it is evident that the wall ties play a very important role in the strength of brickwork. This role is possibly even more important than the amount of mortar, which was the basis of rejection of the earlier brickwork.

The next wall panel to be tested was Panel B (see Figure 20). It was 1550 mm (6.5 bricks) long, 2150 mm (25 bricks) high and had ties at courses 4, 9, 14, 19 and 24. The ties connected the wall to three studs, thus there were 15 ties.

Wall Panel B had virtually no movement at the relatively low design pressure for Melbourne conditions. At about twice that design pressure the maximum movement was 1 mm at the top of the brickwork. This indicated that the ties at the top of the studs were tending to straighten. At this stage the brick leaf and the studs were moving about the same amount at mid height, that is those ties were transferring the load without deforming.

The wall continued deflecting in this fashion until about six times design pressure for Melbourne conditions, when there was a sudden movement of about 7 mm at the top of the brickwork, resulting in a total of about 12 mm. The wall eventually failed in bending at a pressure of 2.0 kPa by cracking of a mortar joint at mid height. Inspection of the ties showed that while most pulled out of the mortar or pulled the nail from the stud, one tie broke in tension at the interface with the mortar.

5.3.2 Original wall panels

The remaining brickwork on the house was of original construction. While there were no other full height panels available for test, there were a number of suitable half height and larger panels between window openings. In each case the panels were approximately 1200 mm long and were supported by ties to three studs. Four such panels of original construction were tested. Their location is indicated in Figure 20. A summary of the results of all the suction test is given in Table 10.

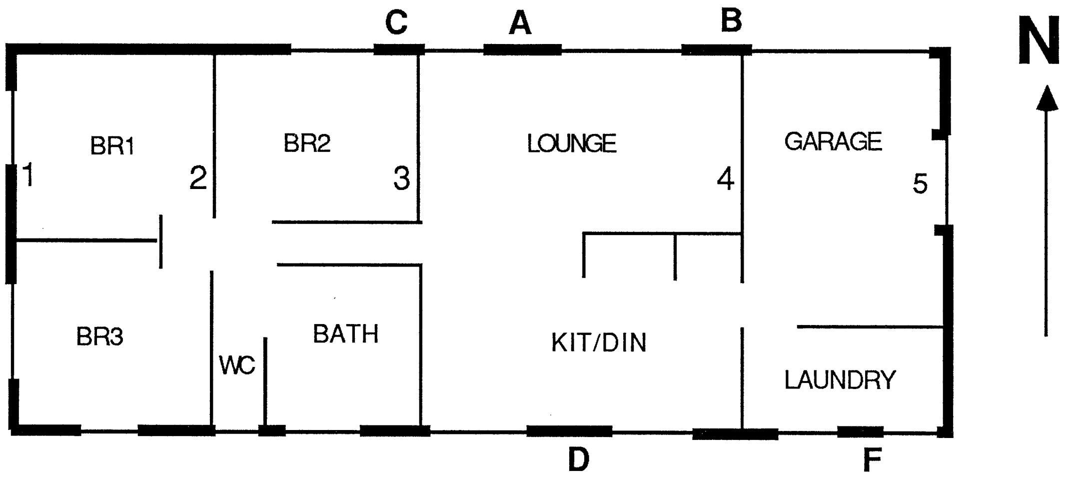


FIGURE 20 Location of Test Wall Panels

5.3.3 Summary of results

Table 10 summarises the test results of the six wall panels subjected to simulated negative pressure loading. Some explanation of the table is needed. As has already been explained, isolated panels A and B both failed ultimately in classical bending and therefore their loads at failure can be expressed in terms of pressure acting on the panel. Because of this failure mode the tensile forces in all the wall ties did not reach a maximum value before the wall failed in bending and thus only those at mid height failed. The failure mechanism of panel C was part bending and part sliding, therefore both equivalent pressure and tensile load at failure have been included. The panel broke at the mortar line at the bottom of the window opening. The pressure to cause bending failure is a little higher than that of panels A and B, reflecting the influence of the sliding mode of failure.

TABLE 10
RESULTS OF SUCTION PRESSURE TESTS ON WALL PANELS

Panel	Nominal Area (H x W) (m)	Failure Load (kN)	Equivalent Pressure at Failure (kPa)	Approximate Tensile Load per Tie (kN)
A	2.15 x 1.3	4.80	1.7	-
B	2.15 x 1.2	5.34	2.0	-
C	2.25 x 0.83	4.50	2.4	0.50
D	1.05 x 1.24	5.12	-	0.57
E	1.05 x 1.34	4.91	-	0.55
F	1.05 x 1.2	4.65		0.52

Although the three smaller panels were tested in a similar manner to the larger panels, the results indicate that they were influenced by the adjoining larger portion of the wall and therefore did not fail in a bending mode. Instead, all of the ties above the lower level of the adjacent window openings pulled out of the mortar bed suddenly, leading to the collapse of the panels. The results are therefore expressed in terms of the average tensile force in the wall ties. The similarity between the three results indicates that this approach is sound.

5.4 Comparison of Strengths of Brick Skins

Table 11 summarises the maximum pressure applied to the brick veneer skin during this series of tests, both before and after rebuilding and under positive and negative pressure loading. It includes only wall panels twenty or more bricks high, therefore excluding the last three small wall panels listed in Table 10. The table also includes, as the last line in the table, the results obtained from lateral pressure tests on the brickwork of a previous brick veneer house tested for cyclone wind conditions.

There are a number of obvious conclusions that can be drawn from Table 11. Sound uncracked brickwork attached to a timber frame has the capacity to resist high positive lateral wind pressures. The strength of brick veneer construction was not as great in resisting negative (suction) pressures as it was in resisting positive pressures, but it was still much more than the design pressure for walls in areas not affected by tropical cyclones. This difference in the wall strength between the two forms of loading reflects the performance of the wall ties. As has already been stated, the shape of the ties used in the construction of the Melbourne house was such that they had a significantly higher compressive strength than pullout strength from the mortar.

TABLE 11
SUMMARY OF LATERAL PRESSURE TESTS ON BRICKWORK

Wall Length (m)	Pressure Direction	Design Pressure (kPa)	Pressure at Failure (kPa)	Failure Mode Initial	Ultimate
1300	positive	0.23	7.6	Unknown.	Ties buckled, bending of b'work
1550	positive	0.23	5.3	Cracking	Bending of b'work
2500	positive	0.23	>2.3	None	(large deflection)
1300	negative	0.23	1.7	Ties pulled.	Bending of b'work
1550	negative	0.23	2.0	Ties pulled.	Bending of b'work
1300#	positive	0.64	1.9*		Ties buckled

Results from a previous brick veneer house tested for cyclone winds
* After having been subjected to 1000 cycles building up to 0.64 kPa.

Despite their important role in the lateral strength of brick construction it appears that there is little planning of the type of wall tie that is used for domestic construction. The tie used for the brickwork in the previous test house, constructed for cyclone areas, was of flat cross section and therefore had a much lower buckling strength than the ties used in the Melbourne house. This is shown by comparing the strength of the last wall in Table 11 with the first three values in the table.

6. COMBINED UPLIFT AND LATERAL LOADING

6.1 Elastic Response of House

This test can be considered the most important in the whole series as it represents the wind engulfing the entire building and applying maximum average uplift pressures coincident with maximum average lateral pressures. The mechanics for applying the uniform uplift and lateral loads have been outlined in Section 3.2. As was explained in that section, the uplift pressure was applied directly to the roof battens rather than to the roof tiles.

Seventy displacement transducers were used to monitor the response of the house for these tests. They were set to measure vertical displacements of the rafters and the top plates and horizontal displacements of the top plates and the transverse walls. Typical locations of the transducers have already been shown in Figure 12.

Two preliminary tests to lower loads were conducted before the house was loaded well beyond its design load. At the design load for Melbourne, during this third test, the maximum vertical movements were both exactly 1 mm measured on the gable rafter at the west end and on a rafter on the garage lintel near the east end. This movement was the lifting of the rafter relative to ground. The lifting at these two locations continued to increase linearly until about four times the Melbourne design load, when the rafter over the garage started to lift at a greater rate. This was probably due to the combination of movement of the truss joint and the lintel tie down strap. That strap was secured to the lintel by four nails driven into the end grain. The maximum pressures applied during this test were 1.2 kPa uplift and 1.68 kPa lateral which were 4.7 times the design value for Melbourne, 3.7 times the value for Sydney and 2.9 times the Brisbane design pressure. At this load combination the maximum vertical displacement recorded was 8.6 mm at a rafter on the lintel beam over the garage. The lintel beam itself lifted 4.5 mm at that position. The next largest deflection was 5.8 mm at the gable rafter on the north west corner. The corresponding rafter on the other slope, and therefore at the south west corner, lifted 3.9 mm which proved to be the third largest movement. The greatest racking deflection of a transverse wall at this high load was only 1.4 mm at wall 2 between bedrooms 1 and 2. The largest overall lateral deflection was 2.4 mm at the top plate in the lounge room.

The loads were then removed and the house was inspected for damage. Although there had been some relatively large deflections, there was no obvious damage to the roof structure, its fixings, the ceiling or to the walls in racking. The nearly 9 mm uplift at the lintel rafter, which was due to the combination of two vertical displacements, settled down again under the weight of the tiled roof. In summary, there was no evidence of the house having been severely loaded.

6.2 Testing to Failure

The next combined uplift and racking test was conducted to determine the overall strength of the Melbourne house and the mode of failure. The combined load was increased in increments, maintaining the correct ratio between the uplift and racking forces. The house responded in a similar manner to the previous test, with the rafter at the lintel beam moving 9.3 mm at the load that caused 8.6 mm in the last test. However, one load increment later the rafter moved upward significantly, to have a total displacement of 20 mm. This failure occurred at the equivalent of 1.33 kPa uplift pressure and 1.86 kPa racking pressure. Table 12 compares this pressure at failure with the average design pressures calculated for Melbourne, Sydney and Brisbane and lists the ratios between design and failure pressures.

TABLE 12
COMPARISON OF PRESSURES AT DESIGN AND FAILURE

Location	Average Design Pressures Uplift (kPa)	Average Design Pressures Lateral (kPa)	Pressure at Failure Uplift (kPa)	Pressure at Failure Lateral (kPa)	Maximum Strength Ratio*
Melbourne	0.25	0.35	1.33	1.86	5.3
Sydney	0.32	0.44			4.2
Brisbane	0.42	0.57			3.2
Region A	0.28	0.39			4.8
Region B	0.40	0.55			3.3

* Based on average roof pressures. High local pressures could halve these ratios.

There were two different but associated modes of failure. One was by the lifting of a batten and the other was by lifting of the rafter. The two were distinct events, but the latter failure was due to a redistribution of load caused by the former one. The roofing batten that lifted was over the garage and therefore in the area of interest because of the larger deflection measured at the nearby lintel. The failure was not associated with that deflection but was just a typical case of the single 50 x 2.8 mm nail being overloaded and withdrawing from the rafter. This nail withdrawal caused the adjacent rafters to be overloaded and their fasteners to start withdrawing from the top plate.

This failure of a batten fixing causes a minor problem in the analysis of the strength of the house. As has been mentioned, battens in some parts of the roof are required to resist much higher than average pressures. The local pressure factor used in design can result in some battens having to withstand twice the average design pressures. Although the batten that failed was not one of the highly loaded ones, it does highlight the potential for failure at ratios of about half of those listed in Table 12. That would result in the test house being considered unsatisfactory for Brisbane and region B.

It was decided to terminate the combined uplift and racking tests at that stage because of the high average strength ratio obtained. Repairs were made to the damaged sections of the roof and the test programme for racking was continued.

6.3 Significance of the Tiled Roof

The mass of the tiled roof played a significant part in assisting this house to resist the simulated uplift wind pressures. Obviously, the heavier the roofing is, the lower is the proportion of uplift force that has to be resisted by the roof structure and its fastenings. In fact, in this instance the weight of the roof tiles and roof structure is greater than the average uplift pressure generated by the design wind for either Melbourne, Sydney or Brisbane. The recommended average mass of a concrete tiled roof is 53.7 kg/sq. m. This can be converted to a pressure of 0.53 kPa, which is greater than any of the design uplift values listed in Table 4, i.e. 0.25, 0.32, 0.42 kPa for Melbourne, Sydney and Brisbane, respectively. Thus when the uplift forces equivalent to those design pressures was applied to the test house, the weight of the tiles and roofing had not been overcome. It was only at a test pressure in excess of twice the Melbourne design uplift pressure that the wind forces started lifting up on the roof structure and its fasteners.

It must be stressed, however, that the conditions stated above apply only to average pressures acting on the whole of the roof structure. Local pressures acting on individual elements of roofing or roof battens can be well in excess of the average values. Calculations show that the wind storm which hits Melbourne at an average frequency of five years generates sufficient local uplift pressure to overcome the weight of a roof tile. These high local uplift pressures form the design case for roof cladding and its fixings. Because of the differences between the average wind uplift pressures that act on the whole roof structure and the high local pressures that can act on some elements, the house testing research programme concentrates on the global forces acting on the house and acknowledges that roof cladding is better tested in the laboratory.

6.4 Effects of Disabling Internal Walls

Chronologically, this final uplift and racking test was conducted after the racking tests described in Section 7, but it is more appropriate to discuss the results in this section. The test was conducted after the diagonal braces in all of the internal walls had been cut and the lining had been removed from most of the internal walls. The aim of the test was to determine if the house could still perform satisfactorily if it had a notional plan as shown in Figure 24.

The uplift and racking forces were increased in their correct ratio up to a failure load equivalent to 1.49 kPa uplift and 2.10 kPa lateral pressure. Displacements were of a similar order to those measured previously. The ramifications of these displacements will be discused in Section 7. From

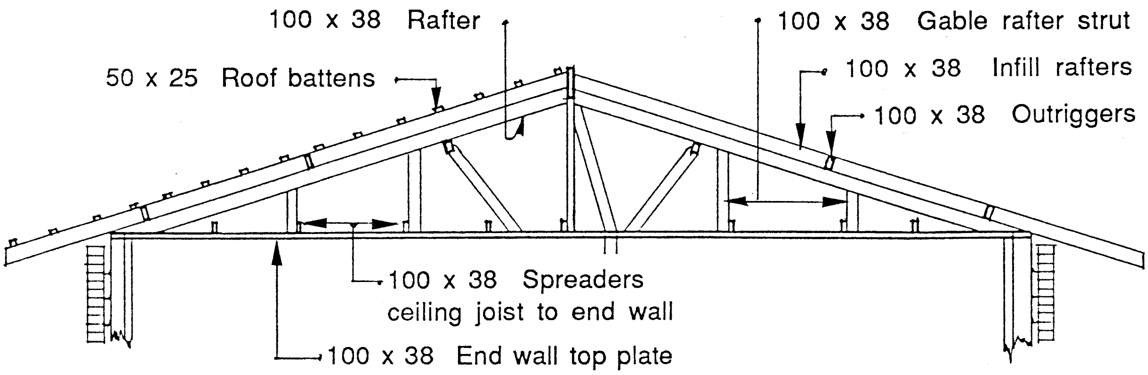


FIGURE 21 Gable End Roof Construction

the strength viewpoint the absence of the diagonal braces and lining on some walls made no apparent difference as the failure was caused by uplift forces only.

The failure was associated with splitting of the outriggers at the gable end roof construction. Details of the gable ends of the roof are shown in Figure 21. For clarity the battens have not been shown on one slope of the roof. To accommodate the verge overhang, two outriggers extended from the first rafter inside the house. In order to maintain the correct roof line the outriggers had to be in the plane of the other rafters in the house and were therefore supported over the end wall by a lower rafter which in turn was strutted at three locations between its heel and the ridge. Nogging pieces were located between the outriggers and were nailed to them. This chain of tie down relied upon the tension strength of the timber perpendicular to the grain as one of its links. With the outriggers spaced up to 1700 mm apart they attracted a significant uplift load which caused two of them to split longitudinally. This failure allowed the blocking pieces to pull free of the rafter and the underpurlins to lift off their supports, which resulted in large deflections. The same type of failure occurred at each gable end.

7. EXPLORATORY WALL RACKING

The object of this series of tests was to investigate the effect of systematically disabling the specified bracing system for the internal walls and later, nullifying them completely by removing the lining material. A uniform racking test was conducted before the programme started and between each step in the dismantling procedure.

The reason for the tests was to investigate the effect of internal wall spacing on the lateral strength of the house. Queensland building regulations specify a maximum spacing of bracing walls of 9 m for non-cyclone areas.

The test pattern was to simulate the racking forces by applying a uniform lateral load to the top plate of the designated windward wall. The lateral response of the house was measured by 43 displacement transducers located mainly along the top plate of each long wall. They were positioned to measure the response of the transverse walls and the long walls midway between the transverse walls. From the previous tests it was anticipated that there would be little measurable response of the house at design pressure. It was therefore decided to apply the load in increments up to 8.1 times the Melbourne lateral design pressure in the first instance and reduce the load for later tests if necessary. For most tests the uplift pressure was not applied.

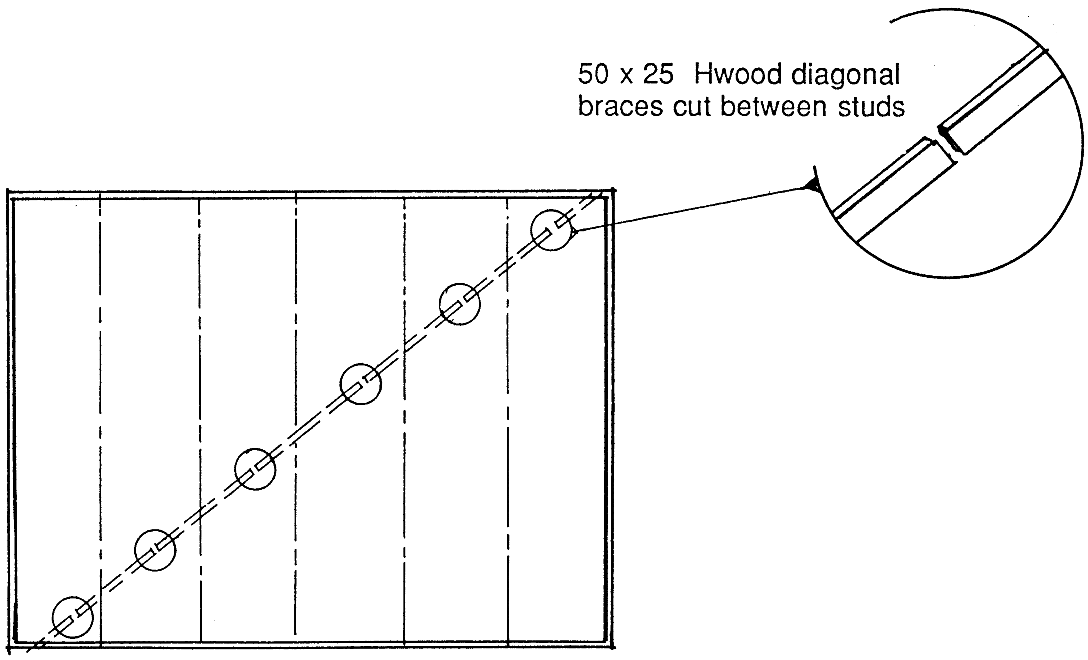


FIGURE 22 Disabling Wall Braces

7.1 Disabling Diagonal Bracing

Almost all of the timber framed walls were braced with diagonal timber members, the only exception was the short kitchen/laundry wall. In this series of tests the braces in each wall were in turn cut between each of the studs and then a racking test was conducted. The studs and braces were located with the aid of a stud finder. As a basis for comparison, a racking test was also conducted before any of the braces were cut. When cutting the braces between each stud, care was taken to ensure minimal damage to the plasterboard. Figure 22 shows the technique. The procedure of disabling the braces in a wall and testing was then continued until all braces had been cut.

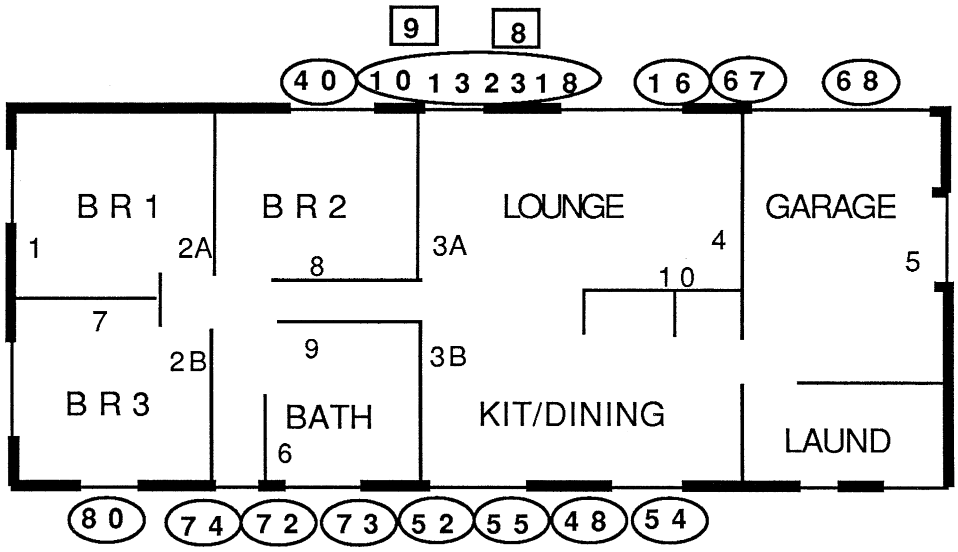


FIGURE 23 Location of Bracing Walls and Important Displacement Transducers for Exploratory Wall Racking Tests

Figure 23 shows the wall numbering system used for these tests as well as the location of the lateral displacement transducers that responded the most during the series of tests on disabling. The numbers in circles represent gauges on the timber top plate and those in squares represent gauges measuring movement at the top of the brickwork. The other numbers represent the walls, 1 to 5 being the main transverse walls, 6 being the bathroom wall and 7, 8, 9 and 10 being longitudinal walls. Walls 2 and 3 have been designated A and B to allow separate reference to each of the diagonal braces in them.

Most of the lateral movement occurred along lintels over the window openings although it should be noted that gauge 8 was measuring movement of the brickwork. The maximum movements were comparatively small, only in the order of 2 to 3 mm. Table 13

TABLE 13
DISPLACEMENTS MEASURED AFTER CUTTING BRACES

Braces cut in wall number	Horizontal Displacements (mm) at Gauge Numbers 40	23	8	18	16	67	68	48
None	2.0	2.2	2.0	1.6	2.3	1.6	2.1	1.0
3A	2.0	1.8	1.5	1.5	2.2	1.3	1.8	0.6
+ 3B	1.7	1.6	1.7	1.4	2.0	1.3	1.7	0.8
+ 2A	1.7	1.6	1.6	1.3	2.1	1.2	1.4	0.8
+ 2B	1.7	1.2	1.6	1.2	2.1	1.5	1.6	0.8
+ 4	1.9	1.3	1.6	1.4	1.9	1.4	1.5	0.9
+ 6	1.8	1.4	1.6	1.3	2.0	1.0	1.4	0.9
+ 7,8,9,10	1.7	1.4	1.7	1.4	2.0	1.4	1.4	0.9

summarises the major displacements measured during the test series. The measured data has been corrected for the effects of residual deflections. The symbol "+" in the left hand column of the table indicates that the braces were cut in that wall in addition to the walls already mentioned in the table. For example, the line marked "+ 2A" lists the results for the case where the diagonal brace had been cut in the northern section of wall 2 as well as in walls 3A and 3B.

Regarding Table 13, the results should be put into perspective in two respects. Firstly, in order to achieve meaningful lateral displacements the house was loaded to 8.1 times its design lateral load for Melbourne conditions. Therefore the displacements at design wind loads would only be in the order of 12% of the values listed in the table. Secondly, the displacement measuring equipment had been developed for significantly larger displacements and therefore had an accuracy limit of 0.2 mm thus the difference between, say, 1.2 mm and 1.4 mm may not be significant. The main information contained in Table 13 therefore can be summarised as follows:

(a) the house was very stiff and very strong under lateral loading

(b) the internal lining material provides a much more effective bracing medium than conventional diagonal bracing.

As can be seen from gauge positions shown in Figure 23, almost all of the major lateral displacements occurred on the windward face. The only significant movement on the leeward face was measured by gauge 48 at the middle of the top plate in the kitchen/dining area wall. This lack of movement on the leeward face was not unexpected as it showed that even without their diagonal braces the transverse walls were still acting as bracing walls and transferring most of the lateral forces to ground. The movement at the kitchen/dining wall illustrated a bending action in the plane of the ceiling between transverse walls.

A point of academic interest demonstrated in Table 13 is the apparent stiffening of the house under repeated loading. The house is generally showing less overall displacement after the braces had been removed than before any were removed. This is a reflection of the repeated loading of a nailed timber structure. Because two tests were conducted for each line shown in the table, more than a dozen racking tests had been conducted before all the values in the table were established. This repeated loading allowed the fastenings to remain to one side of the small slots they had generated during the initial tests. If the loading sequence included a load reversal cycle, the deformations would have been considerably larger than those measured for the later tests listed in Table 13.

7.2 Removing Plasterboard Lining

The next group of tests in the exploratory racking series involved the systematic removal of the internal lining from the transverse walls and testing. For example, the first lining was removed from one face of wall 3A and the house was tested, then the lining was removed from the other face and the house was tested again. This process was continued until the lining was removed from most internal walls. In order to simulate continuity of the ceiling, a strip of plaster 100 mm deep was left at the top of each face. It was still glued to both the cornice and the wall studs and would therefore reasonably simulate continuity of the ceiling as if no wall had been there.

Once again the house was loaded to 8.1 times the design lateral loading for Melbourne wind conditions during each test, to ensure that significant displacements occurred. The major displacements measured during the systematic removal of the wall lining are listed in Table 14. Location of the gauges was the same as shown in Figure 23. The "+" notation has again been used to indicate that the wall conditions represent those stated in addition to the worst conditions previously indicated. The reference test, the first listed in the table, is for all diagonal braces cut but before any plasterboard had been removed. Thus it is a repeat of the results of the last test listed in Table 13.

Table 14 demonstrates very effectively how the internal walls were acting as bracing walls. As each of the transverse walls had its plasterboard removed, the gauges in the vicinity registered greater

TABLE 14
DISPLACEMENTS MEASURED ON THE WINDWARD WALL
AFTER REMOVING LINING FROM INTERNAL WALLS

Wall	Faces Removed	40	9	10	13	8	23	18	16	67	68
		Horizontal Displacements (mm) at Gauge Numbers									
(Reference)		1.7	0.5	-	0.6	1.7	1.4	1.4	2.0	1.4	1.4
3A	1	1.9	0.7	-	0.8	2.1	1.4	1.8	2.2	1.4	1.7
3A	2	2.5	1.5	-	1.0	2.3	1.5	1.9	2.4	1.6	1.7
+ 3B	1	2.8	1.6	-	1.6	2.7	2.0	2.4	2.8	2.0	2.1
+ 3B	2	2.8	1.6	-	2.0	2.9	2.2	2.5	2.8	1.9	2.1
+ 8,9	2	3.0	1.8	-	2.0	3.1	2.5	2.7	3.0	2.3	2.3
+ 6	2	3.8	2.1	2.6	3.0	3.4	2.7	3.0	3.4	2.5	2.2
+ 2A	1	4.5	3.1	3.5	3.8	4.0	3.2	3.4	3.6	2.5	2.2
+ 2A	2	4.9	3.1	3.5	3.8	4.1	3.3	3.6	3.8	2.9	2.6
+ 2B	1	5.7	3.8	4.0	4.5	4.6	3.6	3.7	4.1	2.9	2.7
+ 2B	2	6.2	4.2	4.3	4.9	5.0	4.0	4.2	4.3	3.0	3.0
+ 7	2	7.0	4.7	5.0	5.3	5.6	4.4	4.6	4.6	3.3	2.8

movement, showing that they had been relying on the bracing capacity of the wall. Even gauges 8 and 9 at the top of the brickwork deflected about the same as the timber frame measured by gauges 23 and 10 respectively. (At this stage in the test programme the brickwork was still sound as the tests described in Section 5.3 had not been conducted.)

Virtually removing the internal walls meant that the ceiling must act as a horizontal diaphragm spanning from wall 1 to wall 4, a distance of approximately 13 m. Figure 24 shows the nominal plan of the house with its open area. Even with the house in this condition the ceiling was able to transfer the horizontal forces by acting as a very deep beam between the remaining transverse walls. In doing so, it caused significantly greater lateral displacement at the top plate of the leeward wall. Table 15 lists the displacements measured along the leeward wall during the sequence of removing the plasterboard. The gauge locations and wall numbers have been given in Figure 23.

The degree of bracing contributed by the internal walls can be assessed by comparing the movement listed in Table 15 at gauge 72, located adjacent to the wall 6, and gauge 48, located in the middle of the kitchen/dining wall. In the reference test, with the cladding still on all of the internal walls, gauge 72 barely moved whereas gauge 48 recorded almost the maximum movement. Removal of the lining from the two

TABLE 15
DISPLACEMENTS MEASURED ON THE LEEWARD WALL
AFTER REMOVING LINING FROM INTERNAL WALLS

Wall	Faces Removed	Horizontal Displacements (mm) at Gauge Numbers 80	74	72	73	52	55	48	54
(Reference)		1.0	0.4	0.4	0.8	0.4	0.1	0.9	0.3
3A	1	0.9	0.3	0.6	0.8	0.6	0.0	0.9	0.4
3A	2	0.5	0.3	0.4	0.9	1.1	1.1	1.3	0.3
+ 3B	1	0.6	0.5	0.5	1.3	1.8	1.5	1.4	0.4
+ 3B	2	0.8	0.7	1.0	1.7	2.0	1.6	1.8	0.4
+ 8,9	2	0.8	0.8	1.0	1.8	2.3	2.3	2.0	0.5
+ 6	2	0.9	0.8	1.5	2.1	2.3	2.3	2.0	0.7
+ 2A	1	0.9	1.4	2.3	2.8	2.7	2.9	2.3	0.6
+ 2A	2	1.4	1.7	2.4	3.2	3.2	3.1	2.4	0.7
+ 2B	1	2.6	2.7	3.2	3.9	3.6	3.5	2.8	0.7
+ 2B	2	3.1	3.2	3.6	4.3	3.9	3.7	2.8	1.4
+ 7	2	3.8	4.0	4.6	5.2	4.5	4.6	3.6	2.1

sections of wall 3 contributed more to the movement at gauge 48 as gauge 72 was still supported by wall 6. However as the walls within the bedroom area had their plasterboard removed the displacement measured by gauge 72 increased until it was well in excess of that measured by gauge 48.

Again, putting the results into perspective, maximum displacements of only 7 mm were recorded for the house with a nominal plan as shown in Figure 24, when loaded by racking forces equivalent to 8.1 times the design lateral pressure for Melbourne. These racking forces were also equivalent to 6.5 times the design lateral pressure for Sydney and 5.0 times that for Brisbane. (For Regions A and B the ratios are 7.4 and 5.2, respectively.)

A final uplift and racking test was conducted to determine if the house with its very open plan (Figure 24) could still resist significantly high wind forces. The overall performance and the mode of failure have already been described in Section 6.4. The pressures at which failure occurred was equivalent to 6.0 times the design wind conditions for Melbourne. Just prior to failure the windward wall had a maximum lateral displacement of 15.4 mm at gauge 40, 13 mm at gauges 13 and 23 and 12 mm at gauge 68 and at gauge 8 on the brickwork. On the leeward face maximum displacements of 9.5 mm were measured at gauges 73 and 55, with 8.4 mm at gauge 52 and 7.7 mm at gauges 74 and

48. These greater displacements at lower lateral loads are caused by the uplift pressures removing the benefit of the mass of the tile roof. There would have been a frictional effect between the roof structure and the top plates during the racking tests without uplift loading.

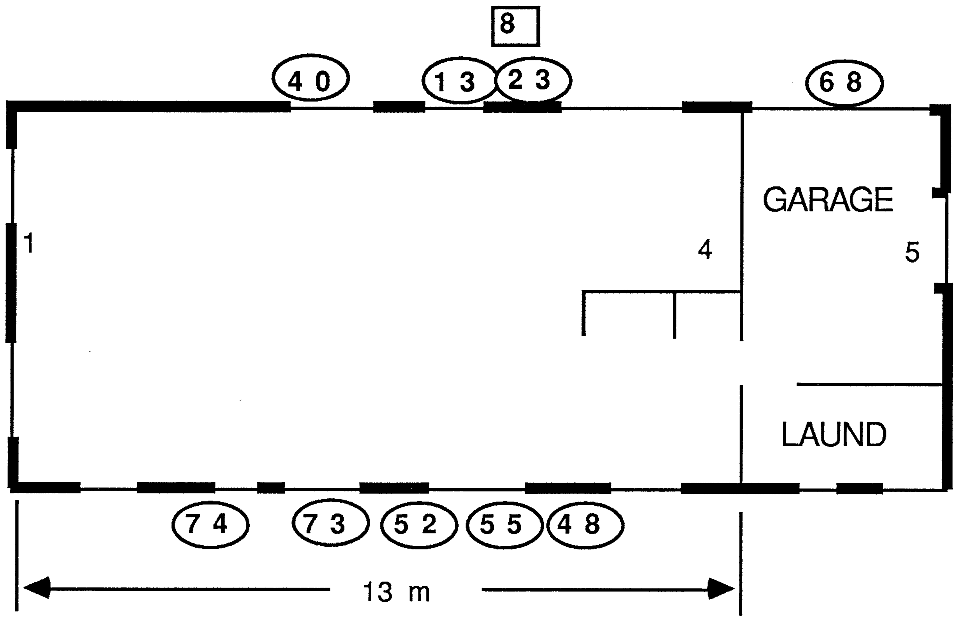


FIGURE 24 Nominal Plan after Removing Internal Wall Lining

8. DESTRUCTIVE RACKING OF REMAINING WALLS

After the disabling tests described in Section 7 only three transverse walls remained intact. As shown in Figure 24 they are wall 1, wall 4 and wall 5. The two external walls were entirely as constructed whereas wall 4 had its diagonal brace cut. It was decided to test each wall in racking to determine its strength.

8.1 Wall 1

Wall 1 was of brick veneer construction. Separate tests were conducted on the two parts of the wall, that is, both the timber frame and the brick veneer skin were independently racked to destruction.

8.1.1 Timber frame

The timber frame, with its plasterboard on one face and diagonal braces in position, was loaded in 2 kN increments of racking force up to 40 kN without any sign of failure. The maximum displacement at the top of the wall adjacent to the point of application of the load was 18 mm. The load-deflection graph was quite linear. However, there was virtually no displacement at the other end of the wall, indicating that the ceiling was attracting some of the load away from the wall.

The wall was then isolated by cutting the ceiling parallel to that wall and disconnecting the noggings that connected it to an adjacent ceiling joist. First cracks in the plasterboard were noticed at 16 kN racking force. They emanated from opposite corners of each window opening. Compression creases formed at the other corners. At 30 kN racking force the wall had displaced 12.6 mm, but as the top plate between walls 1 and 2 had moved 4.5 mm laterally it was thought that some force was still being transferred to the ceiling. This could have occurred through the windward wall top plate loading the ceiling in bearing.

For the final test the top plate in the windward wall was cut and the ceiling removed in that vicinity. The wall was racked again up to 40 kN. There was severe local crushing of the wall near the top plate. The maximum displacement measured was 14.5 mm. At that stage it was decided to terminate the test. One point of interest was that there was no measurable movement of the parallel brick veneer skin during any of these tests. Because of the relative flexibility of the wall ties, it is not surprising that there appears to be no force transfer between the frame and the skin.

8.1.2 Brick skin

The shape of wall 1 with its window openings is shown in Figure 25, but the number of bricks in the figure is not true to scale. To rack the wall a horizontal force was applied to the top course of bricks at the left of the wall as illustrated by the arrow. Failure occurred at a force of 4 kN when the wall cracked through the mortar joints in a generally zigzag shape, initiating from the third mortar bed from the top of the left hand part of the wall. The failure did not continue past the window opening and thus left the rest of the wall quite sound. The failure was very sudden as the wall had virtually no measurable displacement before it occurred.

As the failure was associated solely with the left hand length of the wall, a racking test was then conducted on the central length. This 2270 mm (9 1/2 brick) length of wall was loaded to 16 kN before it failed. Once again the failure was very sudden and appeared to be virtually a tensile failure of the mortar due to the overturning component of the racking force. In this case the crack occurred in the ninth mortar bed from the top.

Both of these loads at failure compare favourably with published design racking loads for single leaf brick walls (Brick Development Research Institute, 1987). The design racking load for a 1200 mm wall is 0.8 kN,

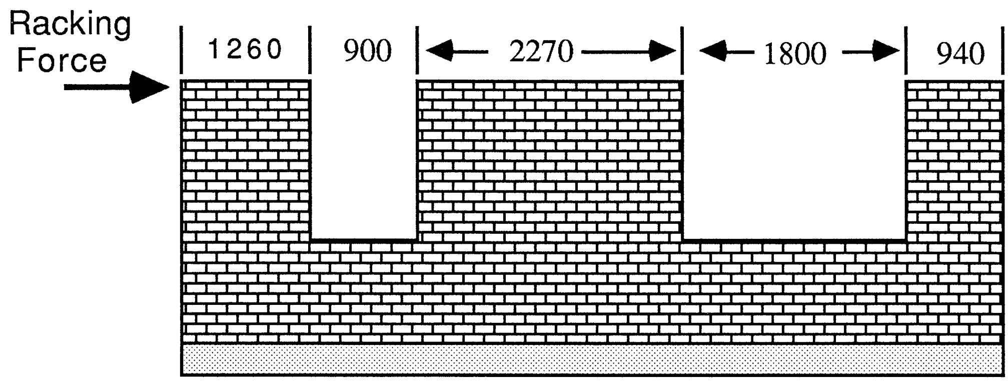


FIGURE 25 Brick Skin of Wall 1 and Initial Racking Position

while that for a 2400 mm long wall is 3.3 kN. The test results demonstrate a factor of almost 5 between design strength and ultimate strength.

8.2 Wall 4

The lessons learned from the tests on wall 1 were put to good use in testing wall 4. The wall was totally isolated from the ceiling before testing commenced. The wall, clad with plasterboard on one face and fibre cement board on the other, resisted a racking force of up to 16 kN without any sign of failure. After that, local cracking and crushing of the lining started to occur but did not develop enough to cause serious failure. In fact, the load-deflection curve was still linear at 34 kN. At that load the plasterboard at the top of the wall near the loading point buckled severely. The gauge near the point of application of the load measured 24 mm, but this would have had a major component due to local crushing of the cladding. The gauge at the other end of the wall measured only 5.7 mm.

Loading was continued until 44 kN when local failure caused the test to be terminated. The top plate and the garage lintel were crushing beneath the loading point and the local crushing of the linings was extending. The true racking strength of the wall would therefore have been a little in excess of 44 kN.

As this wall was not constructed as a bracing wall, it does not have a specific design racking load against which its performance can be measured. By comparison, a designated bracing wall of that length, clad with plasterboard on one face and fibre cement board on the other could have a design racking load of 23 kN, on the simple assumption of the overall strength being the sum of the individual bracing strength of the two cladding materials. Thus the test wall had an implied minimum load factor of approximately 2 although the construction and cladding attachment details of a bracing wall would have been far more positive than were used for the wall tested.

8.3 Wall 5

This brick wall on the east side of the garage was of single skin construction and had four 350 mm square piers, one at each end and the others spaced 1200 mm and 3330 mm from the front wall of the house. The piers were not reinforced nor positively fixed to the slab. The top two bricks on the windward pier had been slightly dislodged due to the lateral loading on the lintel beam during one of the previous tests. The racking force for this test was therefore applied at a level two bricks below the top of the pier, at a height of approximately 2150 mm.

As with the racking test on the brickwork of the west wall, there was no measurable deflection of the wall until failure occurred at a racking force of 22.8 kN. At that force the mortar cracked in the 9th mortar bed from the bottom of the pier, allowing the top portion of the pier to lift. Although the failure was not the classical diagonal failure associated with racking, it was the type of failure anticipated from unreinforced masonry. The cracking of the pier was due to the tensile uplift forces caused by the overturning component of the racking force. Without reinforcement in the pier, the mortar could not withstand these tensile forces.

9. CONCLUSIONS

This Melbourne style brick veneer, tiled roof house performed extremely well when compared with the average pressures likely to occur during a wind storm in suburban (terrain category 3) Melbourne. It also performed well when compared with the average design wind pressures for terrain category 3 exposure in Sydney or Brisbane. Considering the proposed requirements of the draft wind loading code, the performance was satisfactory when compared with terrain category 3 of either region A or region B.

The overload pressures applied to the house indicate that it had the potential to perform satisfactorily in category 2 terrain for Melbourne or region A, but not in that degree of exposure in Sydney, Brisbane or region B. However, it may not perform satisfactorily in Melbourne terrain category 2 exposure if the wind speed is influenced by topographical features such as hills and ridges or from any other feature that may increase the wind speed. Also, to perform satisfactorily in terrain category 2 the battens would have to be fixed better than they were in this house.

It should also be noted that the simulated uplift pressures were applied to the roof battens rather than to the tiles. The following conclusions about the likely performance of the house are therefore based on the presumption that the roof tiles are securely fixed according to the manufacturer's recommendations and will remain on the roof during a wind storm.

Particular conclusions that can be drawn from the test series are as follows:

(a) The Melbourne style house was easily able to withstand the average combined design uplift and racking wind pressures for terrain category 3 (suburban) regions in either Melbourne, Sydney or Brisbane.

(b) Eventual failure of the test house occurred when a batten pulled away from a rafter. The failure caused a redistribution of the applied loads and the probable overload of an adjacent rafter, which led to the rafter lifting off the wall. Failure occurred at 5.3 times the average design uplift and racking pressures for terrain category 3 in Melbourne, 4.2 times the average values for Sydney and 3.2 times the average values for Brisbane.

(c) The test programme did not include application of the high local uplift pressures that roofing and battens on some parts of the roof must withstand. However, as a batten joint caused failure in the test house, consideration must be given to the consequences of these high local pressures. Calculations show that, presuming the tiles remain in position, using the same single 50 x 2.8 mm nails as batten joints would result in the house being considered satisfactory for terrain category 3 in Melbourne or Sydney, but not for Brisbane or terrain category 2 anywhere. The current method of using a single 65 x 2.8 mm grooved nail per batten crossover for terrain category 3 in Brisbane would provide the extra 20% holding power required to make the system satisfactory for that area.

(d) The mass of the roof tiles made a significant difference to the performance of the test house. Their weight was just in excess of twice the average Melbourne design uplift pressure on the roof. Thus if the test house had had metal roofing, its failure load would have been about 3.3 times the average Melbourne design pressures.

(e) The practice of fixing only every third row of the tiles on the roof must be questioned. Calculations show that, allowing for the high local uplift pressures that can occur at the ends and edges of a roof, the weight of a tile would be overcome by the uplift pressures generated by a Melbourne wind having a frequency of about once in five years. For Sydney conditions the weight of the tiles would be overcome by winds occurring more frequently.

(f) Because the average design wind pressures on the face of a building in suburban Melbourne are relatively low, the brick veneer skin was able to resist more than 10 times its design pressure for the case of the uniform wind pushing the brickwork towards the timber frame. One individual panel resisted approximately 25 times the Melbourne design pressure.

(g) Simulated suction pressures on the brick skin caused failure to occur at up to 10.4 times the average design suction (or 5.2 times the local design suction) for suburban Melbourne. This ratio was achieved on brickwork that was not in accordance with the requirements of the Brickwork Code, because of the paucity of mortar on the inner side of the joints.

(h) The strength of the brickwork depended on the wall ties to transfer the lateral forces from the brick skin to the timber frame. The particular ties used in this test house worked extremely well in transferring positive pressure from the brick veneer face to the timber framework and were quite effective in transferring negative pressures between these elements.

(i) Diagonal timber braces had no measurable effect on the racking stiffness of internal walls.

(j) During preliminary lateral loading tests on the house frame the addition of the roofing tiles caused a considerable reduction in displacement of the top of the wall. This was presumably caused by the mass of the tiles increasing the frictional effect of the roof framing on the wall.

(k) During the same preliminary test series the addition of both the ceiling and the cornice reduced the lateral displacement of the top of the wall. The ceiling was acting as a horizontal diaphragm, with the cornice acting as an efficient means of transferring the racking force between transverse walls and the ceiling.

(l) Both the plasterboard and fibre cement internal lining provided all of the bracing necessary to resist the lateral design pressures, although they were fixed in their conventional manner rather than as bracing walls. During the exploratory racking test series the house was loaded repeatedly to 8.1 times its design racking load for Melbourne conditions without any signs of failure.

(m) After nominally removing most of the internal walls, leaving an internal open space of approximately 13 x 7 m, the house was still able to withstand lateral forces of 8.1 times its design racking load for Melbourne conditions without any signs of failure but with greatly increased lateral displacements of the top plate.

(n) With the internal space of about 13 x 7 m the house resisted combined uplift and racking forces of 6.0 times those for Melbourne conditions before failure occurred. The mechanism of failure was by the gable end rafter lifting away from its supporting struts.

(o) There was no evidence of the transfer of racking forces in either direction between the timber framework and the brick veneer skin. The racking stiffness of the brickwork was far too great to receive or transfer forces through the flexible wall ties.

(p) The racking strength of the brick veneer skin was affected by the presence of window openings which effectively isolated the lengths between windows. The racking strength of both the veneer and the garage wall with unreinforced piers was finally dependent upon the tensile strength of the mortar.

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APPENDIX A

This appendix contains the results of bond strength tests on three brick piers laid at the same time as the brick veneer skin walls were first erected. The piers were made in accordance with the provisions of Section 6.6 of the Brickwork Code, that is, they were stack bonded in three groups nine bricks high. The tests were conducted seven days after the piers were laid. Unfortunately, due to lack of experience, the piers were transported from the test site (in the sun) to the structures laboratory. Some joints were damaged during this transportation.

The 1974 edition of the Code specifies a bending test to determine the bond strength of the mortar joints. A more favoured method currently used is the bond wrench test. This latter method allows each joint of the pier to be tested individually, resulting in eight values of bond strength for each pier compared with one from the code test.

In the bond wrench test, a cantilever arm is clamped to the top brick of the pier and the one directly below it is supported. Weights are added to the cantilever at a specific distance along the arm until the mortar joint fails. The test is then repeated on each brick down the pier.

The following table lists the bond strength in bending obtained from each of the tests. The stroke implies that the joint was damaged during transit. The bending strength was calculated by dividing the bending moment by the section modulus of the brick cantilever.

TABLE A1
BOND STRENGTH IN BENDING OF BRICKWORK PIERS

Joint	Bending Strength (MPa) of Brickwork Piers								
	Pier 1			Pier 2			Pier 3		
1	0.30	0.28	0.23	0.27	0.16	0.11	0.28	0.27	0.17
2	0.25	-	-	-	0.35	-	0.40	0.40	0.35
3	0.10	0.32	-	-	0.30	0.24	0.37	0.39	0.40
4	0.36	-	0.34	-	0.19	0.38	0.45	0.46	0.47
5	0.41	0.36	0.30	-	0.29	0.35	0.48	0.43	0.35
6	0.38	0.37	0.20	-	0.38	0.43	0.38	0.47	0.47
7	0.29	0.33	0.31	-	0.32	0.37	0.44	0.50	0.46
8	0.21	0.22	0.27	0.15	0.22	0.15	0.34	0.45	0.33