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SIMULATED CYCLONE WIND LOADING OF A BRICK VENEER HOUSE

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SIMULATED CYCLONE WIND LOADING OF A BRICK VENEER HOUSE

G.F. Reardon

SYNOPSIS

A timber framed brick veneer test house was built according to the provisions of the Queensland Home Building Code for terrain category 3 exposure in cyclone prone areas. Large steel frames were erected to apply the simulated wind loads. During construction preliminary racking tests were conducted to ascertain the change in response by adding the roof and ceiling.

Combined cyclic uplift and lateral loads were applied to the house to simulate the gustiness that occurs during a tropical cyclone. This form of loading caused failure by cracking of the light gauge metal straps tying down the trusses, prior to completion of the loading schedule. The metal straps were replaced by framing anchors and the cyclic load testing repeated. The framing anchors performed somewhat better than the straps although some of them cracked also.

The house was very stiff and strong under lateral forces. The brick veneer skin was unaffected by the schedule of cyclic pressure loading and failure eventually occurred by buckling of the brick ties. The racking response of the house was unaffected by the disablement of internal diagonal bracing. After removal of some internal walls the house still had very high racking strength and stiffness. The brick veneer skin had a high racking strength but there was no evidence of interaction with the timber frame. Internal walls had to be completely isolated before they could be racked to destruction. The single leaf brick wall with engaged piers was extremely stiff and very strong, eventual failure was by torsion at a pier rather than by racking.

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SIMULATED CYCLONE WIND LOADING OF A BRICK VENEER HOUSE

1. INTRODUCTION

Brick veneer construction is the most popular form of housing in Australia. While it may not be the traditional method of house construction in the cyclone prone areas of North Queensland, its use is increasing significantly. Further, it is being used extensively in some of the developing coastal townships in the southern extremities of the proclaimed cyclone prone region (the 50 km coastal strip north of latitude 27^{o}S).

From the domestic engineering viewpoint, brick veneer construction is very interesting insomuch as the outer brick skin is considered to be non-structural and in need of the timber framework for support. This postulation may readily be accepted when considering wind forces acting normal to the brickwork as the single skin would have little resistance to the overturning action of such forces, without the lateral support afforded by the framework. When considering racking forces, however, the concept of the single skin being overturned by forces acting in-plane appears to be far more remote.

This house is the second in a series of full scale research houses built specifically to investigate the load distribution, diaphragm action and general interaction of elements within the complex framework of a house. The first in the series was a high set typical North Queensland house, the overall performance of which has been reported by Boughton and Reardon (1983) and some details have been included by Reardon (1985). A comprehensive assessment of the response of that house will be given by Boughton (-). Other new houses that have been included in the series, Reardon and Boughton (1984) and Boughton and Reardon (1984), were tested on a performance basis rather than for research.

Full details of the construction, the testing and results of the tests will be included in this report. However a detailed analysis of the response of the house has not yet been made. When completed it will probably form the basis of some concise research papers.

2. THE TEST HOUSE

2.1 Design

The test house was specifically designed to be in accordance with the W42 provisions of the Queensland Home Building Code (1982). Thus the design was in accordance with the requirements for terrain category 3 wind conditions of a proclaimed cyclone-prone area, having a design wind speed of 42 m/s. The floor plan was virtually identical to one of the example plans included in the Domestic Construction Manual (1984); the width was made 100 mm shorter to suit an existing floor slab. Figure 1 shows overall details.

In designing the test house full consideration was given to the information already obtained from previous test houses, in both the research and performance groups. Therefore a number of departures were made from the example house in the DCM, so that different forms of construction and materials could be included in the house testing research programme. The main departures are listed below, details of the construction are given in Section 2.2.

(a) Anchor rods were deleted from the external tie-down walls and replaced with light gauge metal framing anchors at every second stud/plate joint, i.e. type T.T.1 as specified in subclause 41.8(2) (a) of the Home Building Code.

(b) Deletion of the anchor rods provided an additional problem in respect of the double top plate construction used for the external walls. In the example house the two ribbon plates were not attached to each other but captured by the anchor rods. Therefore in the test house a 14 x 75 mm power driven screw was used to connect the ribbon plates adjacent to each truss and at joints. This detail is not strictly in accordance with the Home Building Code, as ribbon plate construction is not mentioned in it.

(c) Unseasoned F11 hardwood was used for the external timber frame in lieu of seasoned F5 pine, with an appropriate reduction in cross-section.

(d) The fibre cement bracing panels were deleted from the internal walls and replaced with diagonal metal braces, type T.B.1 as specified in subclause 41.8(3) of the Home Building Code.

(e) The northern window was deleted from bedroom 1.

(f) Corrugated roofing was replaced by metal tiles.

(g) The roof pitch was increased to 15^{o} to accommodate the minimum recommended slope for metal tiles.

Replacement of the anchor rods was a logical step after they had performed adequately in previous tests. The Station had been advised that T.T.1 walls were commonly used for brick veneer construction in Queensland's Sunshine Coast area.

Previous research house had demonstrated the effectiveness of diaphragm bracing walls. Diagonal braces were therefore specifically included to investigate their effectiveness when used as internal bracing walls, clad with lining board fixed in the normal manner.

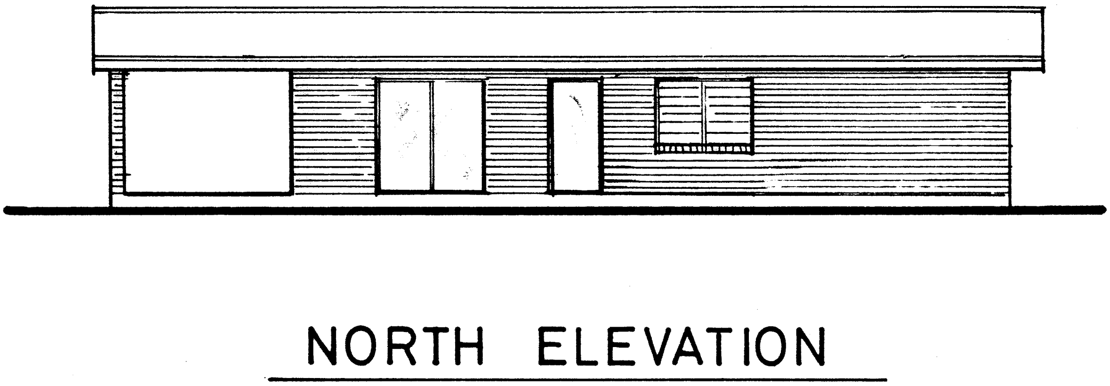
Deletion of the window from bedroom 1 allowed a significant length of full height brickwork on the windward wall. It could be closely monitored during test, without the complicating effects of a window.

All four of the previously tested houses had profiled metal roofing. In each case it acted as a significant bracing medium. Metal roof tiles were chosen for this house because it was anticipated that as discrete elements, they should not have much diaphragm strength. From the test viewpoint their light weight meant that the uplift force applied to the roof structure would not have a large gravity component to overcome.

2.2 Construction

The house was built by a registered builder who is a member of the Queensland Master Builders' Association. He accepted that in constructing the research house he was being asked to use some details that he would normally not use and to use a sequence of construction that no rational builder would follow, e.g. line the internal walls before the roofing was installed or the external walls were built. Most important of all he accepted that his reputation was not under test and he must not use better construction techniques than normal. The house was given the usual set of inspections by the Townsville City Council.

The floor slab was not necessarily built according to the provisions of the Home Building Code insomuch as an existing irregularly shaped slab was extended to suit. Y12 dowels were used to link the new and old sections of slab. At the garage end of the house Y12 starter bars were used to engage in the end piers of the external wall.



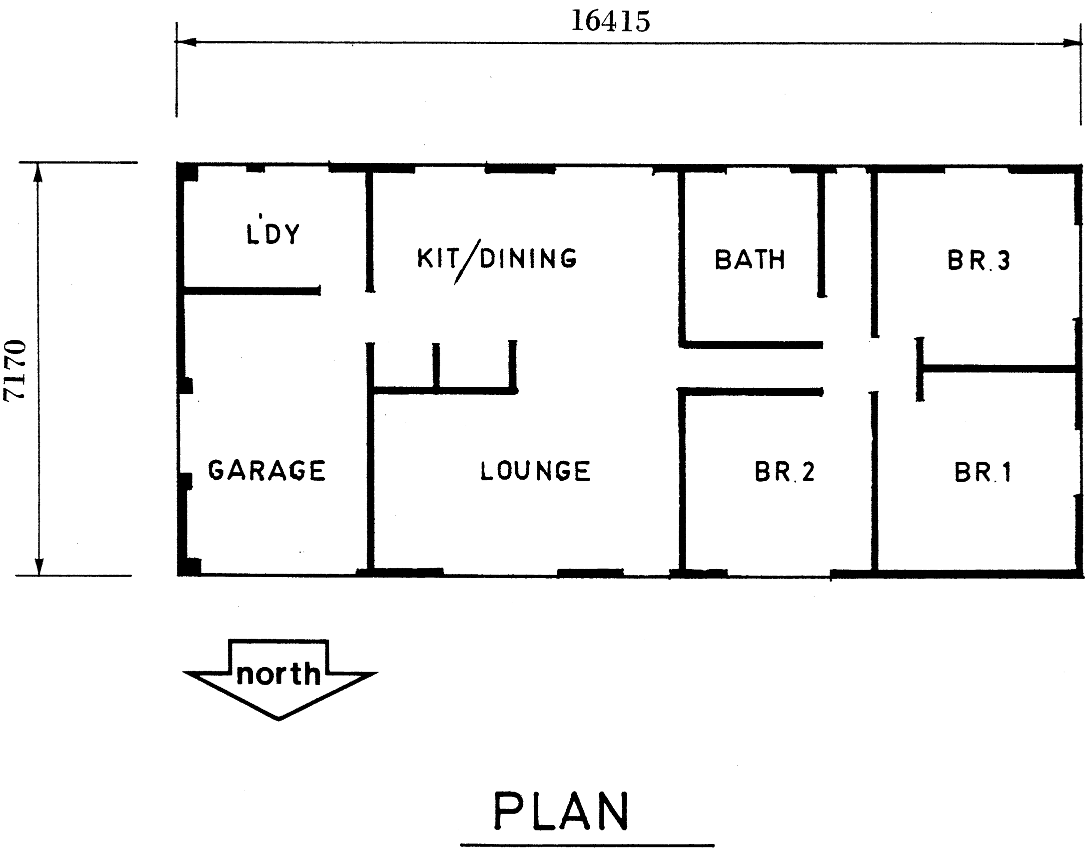


Figure 1 Plan and Elevation of Test House

The slab did not have a step-down edge for the brickwork but this was considered acceptable. As starter bars could not be used M10 chemical anchors were installed at 900 mm spacing to secure the external timber frame to the slab. Similar anchors were used at each end of each bracing wall.

A summary of the main construction details is given below.

Floor slab: 100 mm thick concrete poured over compacted fill. The existing portion had internal beams beneath load bearing walls. Approximately 25% of the total slab was new construction.

Wall frames: External wall frames consisted of 75 x 38 mm F11 hardwood studs at 450 maximum spacings, 75 x 38 mm F11 bottom plate and 2/75 x 38 mm F11 members as the top plate.

Internal wall frames were of 70 x 35 mm F5 pine with studs and plates all having the same cross section.

Lintel sizes and multiple studs were taken from the appropriate TRADAC (1985) recommendations for W42 design.

External Wall fixings: The M10 anchors securing bottom plate to slab were located within 100 mm of a stud. Each alternate common stud was fixed to the top and bottom plates with two framing anchors.

The double member top plate was fastened together with 14 x 75 mm power driven screws at approximately 900 mm spacing and at butt joints.

Bracing walls: The location of bracing walls is shown in Figure 2. Wall 1 was braced with two lengths of 4 mm F14 plywood, one 1.2 m long and the other 1.8 m long. Each was fixed according to specifications for TB4A walls from the Home Building Code, having 30 x 2.8 mm nails spaced at 150 mm centres around the perimeter and at 300 mm spacing on intermediate studs. Wall 1 had a design bracing strength of 6.7 kN. Walls 2, 3 and 4 each had crossed metal bracing fastened with two 30 x 2.8 mm nails at each plate and each stud crossover, that is, Type TB1 from the Home Building Code. Each wall had a design bracing strength of 2.4 kN.

Wall 5 was of single leaf brick construction with four 350 mm square piers, the end two being reinforced with a Y12 rod. The wall had a design racking strength of 8 kN.

The top of each framed bracing wall was indirectly connected to some roof trusses using details specified in the Home Building Code. Figure 3(a) shows the details used for the cases when the bracing wall was parallel to the trusses and Figure 3(b) shows details for the walls transverse to the trusses.

The aim of these details is to allow horizontal forces to be transferred from the wall and the ceiling, while still providing the necessary vertical gap between the trusses and the top of the walls.

Roof trusses: The roof trusses were prefabricated and jointed with toothed metal plates. They were of W-brace construction, spanned 7.17 m, had 15^{o} pitch and were spaced nominally 900 mm apart. The timber was stress grade F14, with all members planed smooth. Top chords were measured as 90 x 34 mm, bottom chord as 70 x 34 mm, the short brace as 93 x 34 mm and the long brace as 70 x 34 mm.

Battens: Roof battens were 38 x 50 mm spaced 365 mm apart and ceiling battens were 45 x 35 mm spaced 450 mm apart. The timber was all F11.

Roof fixings: Trusses were secured to wall plates with looped metal straps, as defined in the Home Building Code. They were 30 x 0.8 mm perforated metal straps taken over the top of the roof truss, looped under the top plate and up the inside face where it was fastened with two 30 x 2.8 mm nails. Figure 4 shows the detail. The trusses in the garage were attached to the deep lintel over the doorway with two framing anchors, as would be the case in practice.

Roof battens were attached to the trusses with two 75 x 3.15 power driven grooved nails per crossover. Ceiling battens were attached with one nail per crossover.

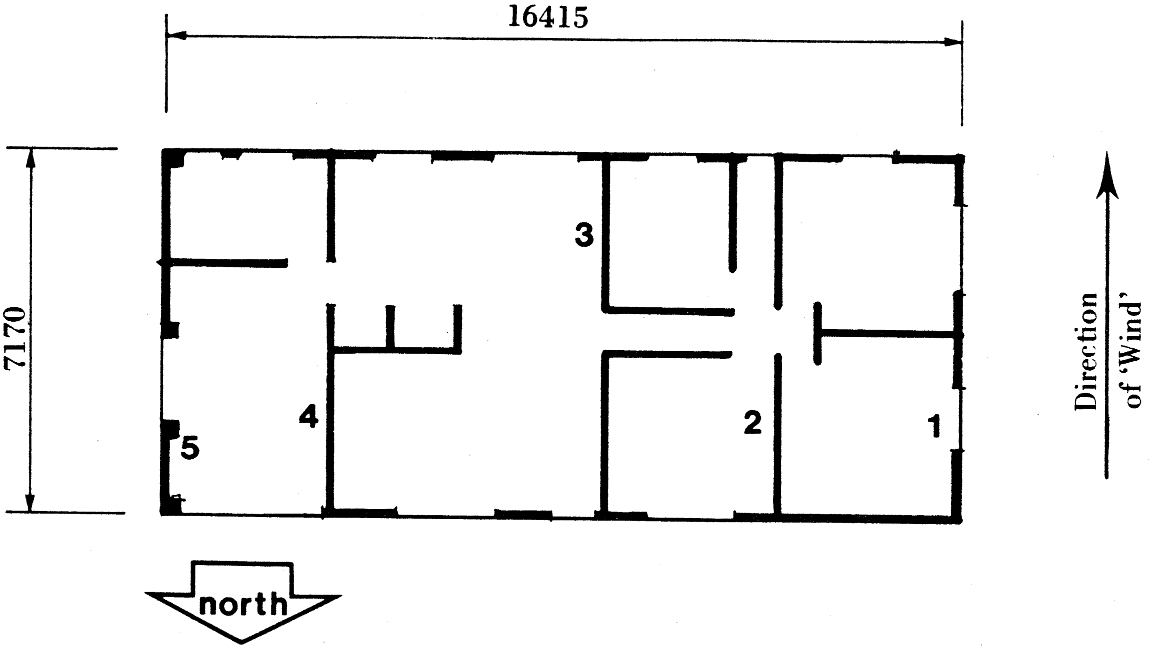
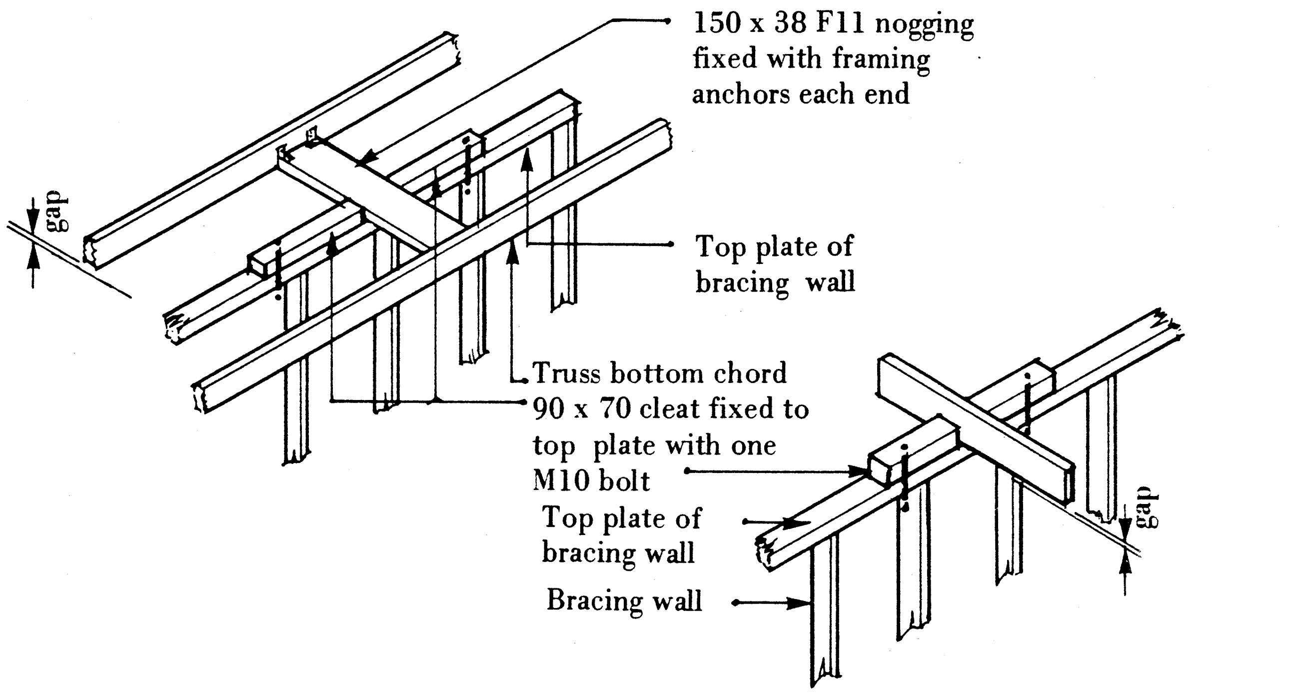


Figure 2 Location of Bracing Walls



(a) Truss parallel to wall

(b) Truss perpendicular to wall

Figure 3 Bracing Wall Connections to Trusses

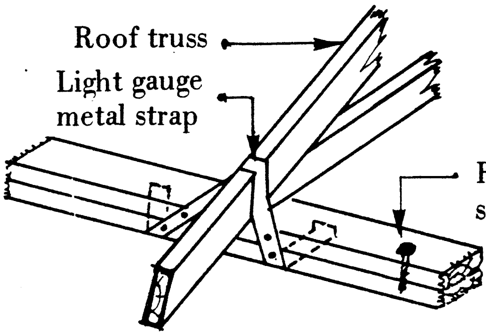


Figure 4 Looped Metal Strap as Truss Tie Down

Brickwork: Standard metric clay bricks were used for the construction. Each brick had ten holes to assist the bond with the mortar. 25 x 0.8 mm flat galvanized steel brick ties were specified at every fifth course on each alternate stud, and every third course around openings. A damp proof membrane was used between the bottom brick course and the floor slab. The mortar was mixed from local ingredients.

Test piers of brickwork were made during construction and the mortar joints were subjected to a bond wrench test after seven days. A full report on the brickwork and mortar was written by the Brick Development Research Institute. In part it shows that the brick ties were often at irregular spacing.

Internal linings: Most walls and ceilings were lined with 10 mm gypsum plasterboard fixed in accordance with the manufacturer's specification for normal use.

Horizontal joints between sheets were finished with jointing cement and a "Broadline" cornice was used at the top of all walls.

In the bathroom 6 mm fibre cement sheeting was used. It was fixed horizontally, the joints were sealed with finishing cement and the same cornice was used. In the garage and laundry however, 4 1/2 mm fibre cement boards were installed vertically. The sheets were butted together on a stud and no jointing cement was used. Timber quad cornice was fitted.

Only 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 when calculating the horizontal wind forces acting on the walls. For example the lintel beam in the garage was loaded during test with the equivalent reaction that would be generated from wind blowing on a large garage door.

3. LOAD SIMULATION AND RESPONSE MEASUREMENT

3.1 Determination of Wind Loads

There are two acceptable methods of estimating the wind loading on a building, by using the SAA Wind Loading Code or by using wind tunnel tests. The latter method would rarely be used for individual housing as its cost could well be a significant percentage of the overall cost of the building. In fact the majority of housing does not even have the luxury of individual design from the Wind Loading Code, but relies on the design of individual members and joints to the appropriate codes and the tabulation of these details in such documents as the SAA Timber Framing Code (SAA 1979), TRADAC manuals and the Home Building Code.

Because codes are general in nature and aim to cover a wide variety of combinations they can be conservative in some instances. This is quite acceptable for structural design, but for the case of the research house it was decided to compare the recommendations of the code with the results of wind tunnel tests.

3.1.1 Using the Wind Loading Code

The specifications of the Wind Loading Code in respect of gable ended houses for cyclone prone areas are quite straightforward.

The basic design wind velocity of 63.25 m/s is modified for terrain category 3 and 6 m height to become 42 m/s. This converts to a free stream dynamic pressure of 1.06 kPa. Pressure coefficients for walls are tabulated in Table B1.1 of the code and for roofs are tabulated in Tables B2.1 and B2.2. Internal pressure coefficients are listed in Table B4 of the code.

As the brick veneer research house could only be tested to destruction for one set of parameters, due consideration had to be given as to which set were considered to be the most critical. This decision mainly related to the assumed direction of approach of the cyclone wind gusts relative to the house. The code specifies that for wind acting parallel to the ridge of a gable ended house the first three or four trusses would have very high uplift forces acting on them. This force would gradually reduce on subsequent trusses until it was very small near the central and subsequent trusses. Conversely for the case of wind acting perpendicular to the ridge line the code recommends uniform but different uplift pressures on each slope. For a roof of 15^{o} pitch these pressures would be only about 50-60% of the pressures on the end trusses for wind parallel to the ridge, but the total uplift force on the entire roof area would be greater.

The latter case, of wind acting normal to the ridge line, was chosen for test mainly for the following reasons:

(a) all 19 trusses had a chance of failing, not just the three end ones,
(b) any additional stresses caused by racking forces would be imposed on the truss tie-down detail,
(c) constraints from loading equipment.

Probably the most contentious issue in the design of low rise buildings for wind is the choice of an appropriate internal pressure coefficient. The Cyclone Testing Station has always maintained that the designer should use the maximum internal pressure coefficient when designing for cyclone prone areas. This recommendation is based on the assumption that windows, with glazing in accordance with the appropriate code, will generally be able to withstand the wind pressure but not the impact of flying debris. Thus a window broken by flying debris would allow the inside of the house to be fully pressurized.

Figure 5 illustrates the pressure coefficients selected from the Code as being appropriate for the research house.

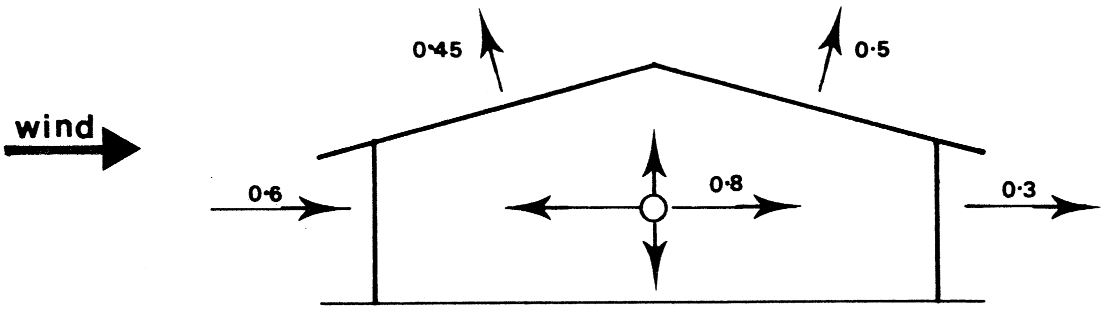


Figure 5 Pressure Coefficient from Code

3.1.2 Using wind tunnel results

Fortunately wind tunnel results were available from models of houses having dimensions very near those of the test house. Two sets were available, one for a house 14 m x 7 m with a 15^{o} gable roof and the other for a house of similar dimensions with a 10^{o} gable roof. It was considered that the generally greater dimensions of the test house may have some effect on the mean pressure coefficients measured in the tunnel, but that this would be no more than a 5% increase.

The wind tunnel tests on the first house (Best and Holmes, 1978) yielded a pressure coefficients of 0.52 on the windward wall for the wind acting normal to that wall. In the other tests (Roy and Walker, 1984) total forces on the model were measured, thus pressure coefficients were not available as such. However the total forces can be converted to force coefficients which are analogous to the sum of pressure coefficients acting on parallel surfaces (e.g. a pressure coefficient of +0.6 on the windward wall combined with a pressure coefficient of -0.3 on the leeward wall would yield an equivalent force coefficient of 0.9). A summary of the pertinent wind tunnel results is given in Table 1.

Tests on the model house with 15^{o} pitch showed that maximum horizontal and uplift forces occurred when the wind approached at an angle of 60^{o} to the ridge line of the house. These results are also included in Table 1.

TABLE 1

Summary of Wind Tunnel Results

	Model with 10^{o} roof pitch	Model with 15^{o} roof pitch	
		Wind normal to ridge line	Wind at 60^{o} to ridge line
Pressure coefficient on windward wall	0.52	-	-
Total horizontal force coefficient	-	0.68	0.89
Total uplift force coefficient on roofing	-	0.57	0.64

3.1.3 Nominal test loads

In respect of pressure on the walls, the values listed in Table 1 can be considered quite compatible with those calculated from the Wind Loading Code. The total force coefficients at 60^{o} is almost exactly the code summation of 0.9. The measured windward pressure coefficient of 0.52 is less than the code value, shown in Figure 5, but the slightly different geometry could account for portion of that. It was decided therefore that the horizontal design pressures be based on the code recommendations for pressure coefficients, i.e. +0.6 on the windward face and -0.3 on the leeward face.

Selection of design uplift loads proved to be more of a problem, as the wind tunnel results were considerably higher than the code recommendations. After some deliberation it was decided to use the values from the code as they would have been the basis on which the house components would have been designed.

However when assessing the final performance of the house in uplift, due consideration would be given to the likelihood of higher uplift loading as indicated by the wind tunnel tests.

Uplift pressure coefficients of -0.45 on the windward slope and -0.5 on the leeward slope were therefore selected as the basis for calculation of the nominal uplift pressures. However these coefficients eventually had to be altered slightly as outlined in Section 3.3.

3.2 Application of Loads

The vertical loading system consisted of twelve large frames, six spaced evenly on each long side of the house. Hydraulic rams were used to apply the uplift loads. Horizontal loading was also applied by means of hydraulic rams attached to large horizontal RHS members spanning between the vertical loading frames. All the rams were linked to the same hydraulic pump but controls allowed the horizontal pressure to be different from that in the vertical rams. In this way the two sets of rams could be used separately or simultaneously.

During the test programme four different loading systems were used:

- Combined uplift and racking
- Uniform racking
- Individual racking
- Uniform lateral 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 to a predetermined force measured by an electronic force transducer in series with the cable. For static tests the load was increased incrementally with displacement measurements made at each increment.

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. Figure 6 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 13.2 m^2, that is, the 4.54 m length of roof slope multiplied by the 2.9 m spacing of loading frames. Each load spreading set reduced the applied load to eight equal portions which were then each distributed to the underside of four adjacent roof battens by means of stiff beams. Thus there were 32 individual loading points for each of the twelve uplift load frames.

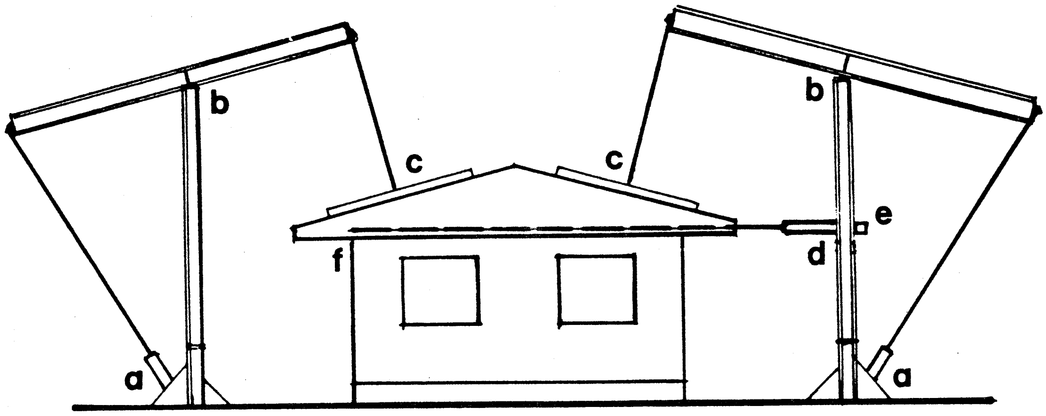


Figure 6 Loading System

It should be noted that the uplift loading was applied directly to the roof battens rather than to the underside of the roofing. This decision was made because of loading constraints, but it is accepted that the performance of metal tiles can be better assessed in the laboratory where a more realistic simulation of uniform load can be applied to them.

Figure 6 also illustrates the system used to apply the horizontal racking forces. The four horizontally mounted hydraulic rams (d) were attached to a large RHS steel beam (e) fixed to the uplift loading frames at wall height. A cable extended over the ceiling from the ram to a load spreading system (f) at the top of the windward wall. Each ram load was distributed to four loading points approximately one metre apart. There were therefore sixteen loading points simulating uniform loading.

When applying a predetermined uniform racking force to the top plate the force was calculated as being 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.

Combined uplift and racking forces were applied during the cyclic loading sequence and during some of the exploratory tests investigating the bracing performance of walls.

3.2.2 Uniform racking

When uniform racking forces only were applied to the test house, the method of application was exactly the same as for combined uplift and racking. This method of loading was used for some tests during construction of the house and during the investigation of the bracing performance as well as in combination with uplift forces during the cyclic loading sequence.

3.2.3 Racking individual walls

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

To rack a wall a ram was aligned slightly to one side of the line 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 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.

3.2.4 Uniform lateral loading

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

A decision had to be made as to which was likely to be the weaker direction for loading the brickwork. It was decided that the buckling strength of the 30 x 0.8 mm ties, acting as 50 mm long columns, would probably be lower than the bond strength between mortar and tie or the pullout strength of the two

30 x 3.15 mm nails securing the tie to a hardwood stud. It was decided therefore to simulate loading of the windward brick wall by applying a uniform positive pressure to the surface.

Figure 7 illustrates the loading device used to apply the uniform pressure. The stiff frame was hinged to the edge of the floor slab and had a cable attached to its top. A large air bag was inflated between the hinged frame and the brick wall, but without applying pressure to either surface. The pre-determined uniform pressure was applied by pulling the cable with a calculated force, by means of the hydraulic ram, and squeezing the passive air bag against the brickwork.

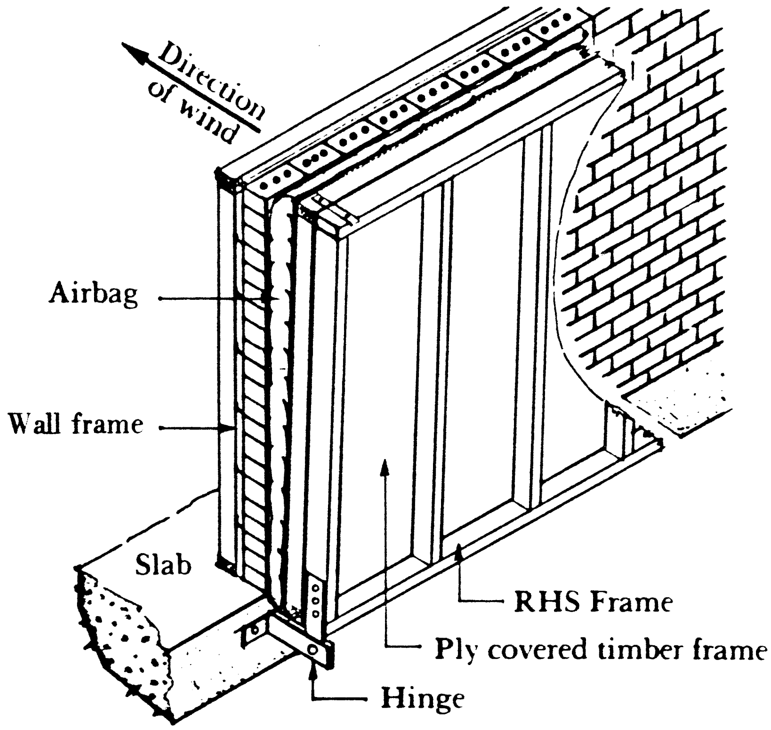


Figure 7 Showing hinged loading frame and air bag.

Five large frames, approximately 2.5 m wide were used to load the brickwork. Where there was a window opening an infil panel was inserted to simulate the window and transfer the applied force back to the timber frame. However this system was not used for the doorways.

The pressure calculated for the brickwork was based on the estimated wind on that face only, not on the sum of windward and leeward pressures as is used for the structural wall frame.

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 should be kept to a minimum, but if they are unavoidable an accurate assessment of their likely effect should be made. It is readily acknowledged that a uniform pressure is difficult to simulate exactly, but application of a series of small point loads is an acceptable alternative. This system has been used for these tests.

The main deviation that had to be made because of equipment constraints related to the uplift pressures acting on the roofing. The loading equipment had been designed to apply uniform pressure to the entire roof surface, not different pressures to each slope as was required by the code and outlined in Section 3.1.3. After some deliberation it was decided to use the truss reaction forces as the basis of equalization of the pressures. Design test load was taken to be that combination of pressures which caused truss reactions of 5.39 kN, the average of the reactions calculated for the code case. This compromise led to a decrease of 6% at the windward wall reaction and an increase of 6% at the leeward reaction. These deviations are considered to be acceptable, but should not be overlooked in the final analysis of the test results.

This design test load can be expressed as having a uniform pressure coefficient of -0.5 on each roof slope, as shown in Figure 8, but this necessitates neglecting the contribution of the forces on the eaves overhang at each end.

A further constraint from the loading gear related the position of uplift roof forces relative to the trusses. Extreme care was taken to ensure that the uplift frames were located where they would apply a uniform uplift pressure to the roof surface. This was achieved as described in Section 3.2.1, but the net effect was to produce twelve lines of loading from ridge to eaves along each slope. Figure 9 illustrates the uplift loading system and Figure 10 shows the layout. Because the line loads were at 1.45 m spacing and the trusses were at 900 spacing, the loads were sometimes applied to battens close to a truss. The worst case was at truss 14 (numbered from the west) where the line load was 90 mm from the truss. Calculations show that this truss may have received 12% greater load than the average and the trusses next to it would have received too little load.

The only other loading constraint related to the frequency of cycling when

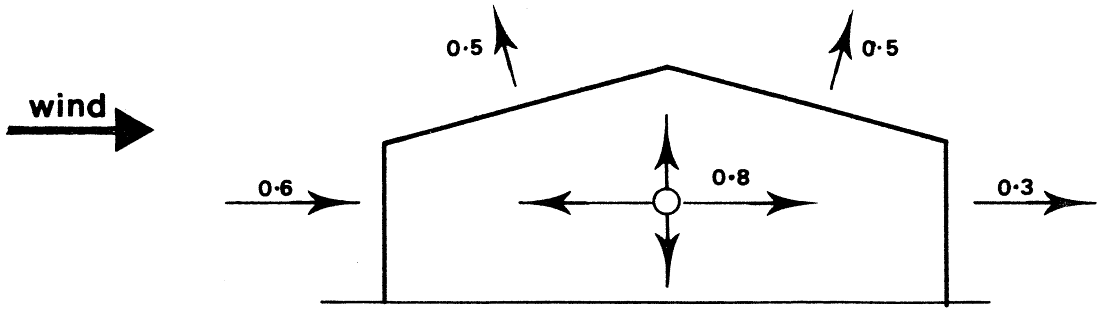


Figure 8 Equivalent Pressure Coefficients

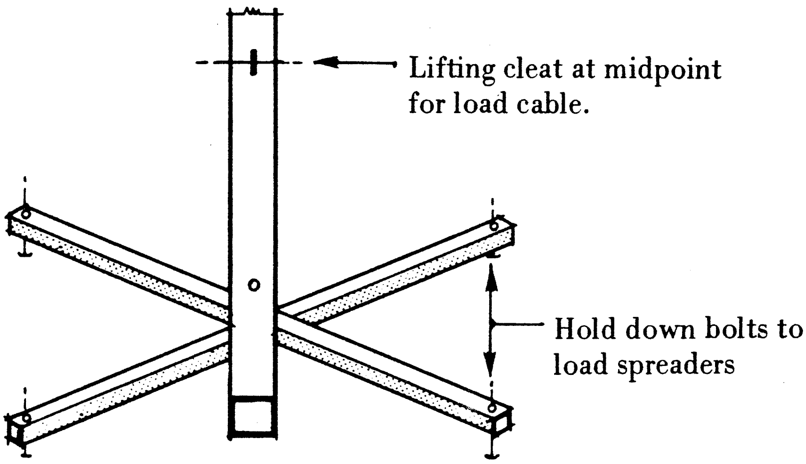


Figure 9 Uplift Loading System

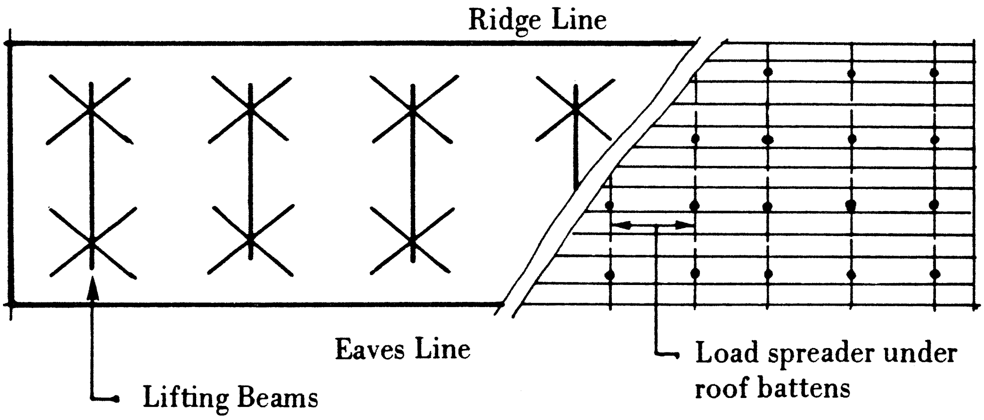


Figure 10 Layout of Uplift Loading

simulating the gustiness of a tropical cyclone. Despite the 10 horse power motor on the hydraulic pump used to control the loading sequence, the frequency of loading was approximately an order of magnitude slower than in a real tropical cyclone. One cycle of load and unload took approximately 30 seconds to complete. This slower cycling rate should not affect the performance of the structural components unless it were to underestimate a case where the real wind gusts approached the natural frequency of the component. This is considered unlikely although there is little information available about the problem. Jancauskas, Walker and Doull (1987) are currently conducting research into the problem of fatigue characteristics of winds on houses. The results of that work should supply much needed information.

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 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 racking and uplift tests vertical displacements of the ends of the trusses and of the adjacent top plates were measured as well as the horizontal movement of the top plates. All displacements were measured relative to sets of independent scaffolding. Thetransducers were fixed to this datum by magnetic base stands, which allowed easy portability from one location to another if necessary. Figure 11 depicts the displacement measuring system.

The load and deflection data was stored by the micro computer and also transferred to magnetic tape. During the course of the test, the deflection at any point could be plotted against applied load and so determine the likelihood of yielding structural components.

4. NON-DESTRUCTIVE TESTING

During various stages of construction of the house the transverse walls were racked slightly to determine their change in response as extra cladding was

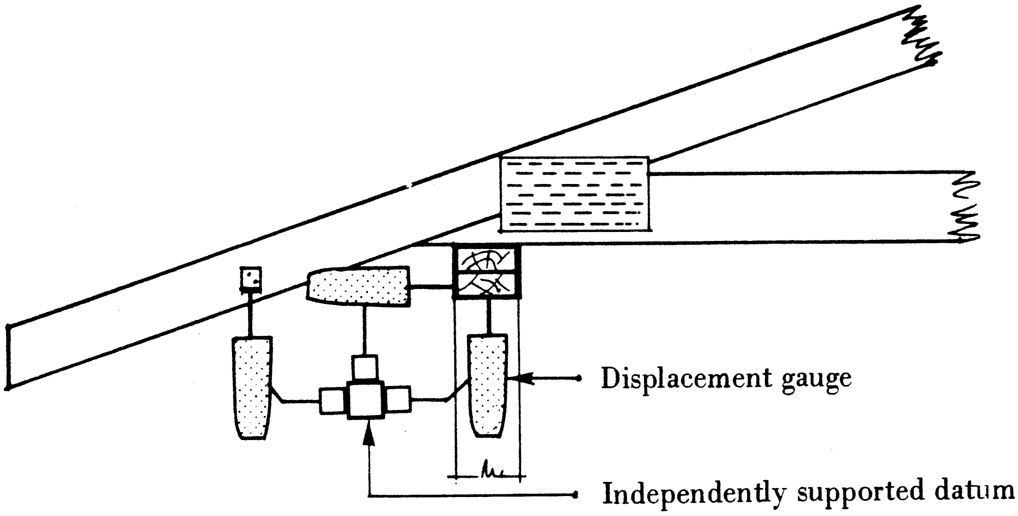


Figure 11 Location of Displacement Gauges for Cyclic Loading Tests

installed. The racking force was applied as a concentrated load to the top plate of the windward wall, as close as practicable to the line of the bracing wall. A uniform racking force was also 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.

The first series of tests was conducted after the timber frame had been assembled, the roof trusses installed, all bracing fitted and the internal lining attached. At the completion of those tests, the roof battens were laid and the metal roof tiles were fitted. The second series of racking tests was then conducted. The ceiling battens and lining were then installed prior to commencement of the third series.

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.

As is often the case in house construction, the internal bedroom walls were aligned on each side of the hallway, as shown in Figure 1. This geometry would allow the possibility of a contribution to the racking strength from the non-bracing wall.

For convenience, the bracing walls were numbered from the west as shown in Figure 2. Wall 1 had a total of 3 m of plywood bracing having a design value of 6.7 kN. Walls 2, 3 and 4 were each braced with diagonal flat steel, having a design value 2.4 kN.

4.1 Racking of Walls Only

In the first instance each wall was racked individually with a force increased incrementally to design load. While this caused an appropriate response in wall 1 (with a design load of 6.7 kN), there was hardly any response in walls 2, 3 or 4 (design load 2.4 kN). The latter walls deflected only 1/2 - 3/4 mm. It was decided therefore to load the latter walls to three times design load to generate a significant response.

These first tests demonstrated that the stiffness of the internal walls was not a function of their design load. It was obvious that the lining material, either plasterboard or fibre cement board, which was glued to each face of the frame was a far more significant factor in the individual stiffness of walls 2, 3 and 4 than the diagonal bracing. There were two reasons why the lining was relatively less effective for wall 1. Firstly the plywood bracing itself had significant strength and stiffness and secondly the plasterboard was glued to one face only of the wall.

By far the greatest response to the uniform loading occurred at the midspan of the lintel beam over the doorway of the lounge, between walls 3 and 4. A deflection of approximately 7 1/2 mm was measured at only half the uniform design racking load.

4.2 Racking of Walls with Roofing

On average the racking tests of each wall, after the addition of the roof battens and metal roofing tiles, resulted in less overall deflection than without the roofing. This could be interpreted as the roofing acting as a diaphragm to disperse the applied force by about 15%. However before making such a conclusion some other aspects of the tests should be considered. The average overall deflection of the walls with the roofing was about 3 mm, compared with 3.5 mm without the roofing, at a load of 3 times design load. As the absolute value of the change was so small, it could have been caused by other factors such as the apparent stiffening of a nailed timber structure after initial loading.

By contrast, the presence of the roofing made a significant change in lateral deflection of the lounge room window head under uniform lateral loading. The beam deflected only about 20% of its deflection without the roofing.

4.3 Racking of Walls with Roofing and Ceiling

The addition of ceiling battens and ceiling caused a further reduction in racking displacement of the transverse walls of about 15%. Once again the main decrease in deflection occurred at the window heads where the measured value was approximately 35-40% of that measured in the previous test. The measured deflection of the lounge room window head beam was less than 10% of its deflection measured without roofing and ceiling.

It was obvious therefore that both the roofing and ceiling had the capacity to provide lateral bracing for the flexible window heads and top plates between walls. The effect of the ceiling and roof diaphragm action on the individual bracing walls was far less significant, but this was to be expected as the stiffness of these walls was considerably higher than would have been expected from the specified bracing elements.

5. TESTING THE BRICK VENEER SKIN

Details of the test rig devised to apply a uniform lateral 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. A design pressure of 0.64 kPa can be calculated for that situation.

It was considered unnecessary to apply vertical loading to the roof structure during the test on the brick veneer skin. Such vertical loading would not be transferred directly to the brickwork and its effect on the interaction between the veneer and the timber framework was considered to be of a secondary order.

To facilitate measuring the displacement of the brickwork and framework, the internal lining had been left off the windward wall frame, and the inside face of the brickwork had been cleaned. It is considered that neither of these actions would have a serious effect on the performance of the wall.

5.1 Cyclic Loading

To simulate the effects of gustiness within a tropical cyclone the brick veneer wall was subjected to a series of load/unload cycles. The loading was conducted in accordance with the recommendations of Technical Record 440 by the Experimental Building Station (1978) [now the National Building Technology Centre]. These industry accepted recommendations require the following loading regime for walls in tropical cyclone areas.

800 cycles	0 - 5/8 design pressure - 0
200 cycles	0 - 3/4 design pressure - 0
20 cycles	0 - design pressure - 0

As was anticipated, the 800 cycles of 0.40 kPa had little effect on the wall, the maximum displacement being 0.7 mm at the top course of bricks in the middle of bedroom 1. At the completion of the 200 cycles to 0.48 kPa, the same brick had increased its displacement to 1.2 mm. The next highest displacement was 0.7 mm measured on the top course three bricks nearer to the end wall. On completion of the total cyclic loading the first brick was displacing elastically 1.8 mm. The greatest movement of a stud was 0.8 mm at mid height. There was no evidence of any displacements being transferred to the leeward side of the house.

5.2 Static Load to Failure

On completion of the cyclic loading regime the windward wall was loaded incrementally to failure. The wall remained very stiff until a pressure of approximately 1.2 kPa (1.9 times design pressure), having maximum displacements in the order of 2.5 to 3 mm. However as the pressure was increased slightly there was a sudden increase in deflection with movements up to 5 mm. Although the wall was still able to resist increases in applied pressure the displacement increased significantly until a maximum pressure of 1.89 kPa was recorded. This pressure is equivalent to 2.95 times design pressure. The largest deflection recorded prior to failure was almost 17 mm, but after failure the top of the brick skin at the two short lengths of brick wall in the lounge room were leaning on the timber frame.

Failure of the brick wall had occurred by a rigid body rotation about the floor slab. Figure 12 illustrates the mode of failure. The brick ties at the top of the wall had buckled severely and those on lower courses showed progressively less buckling of the bottom tie. There was no evidence of movement of ties within the mortar nor of any cracking of the mortar joints, except at the bottom joint where the failure occurred. Also, there was no evidence of lateral translation of the brickwork on the surface of the slab, despite the presence of the damp proof membrane.

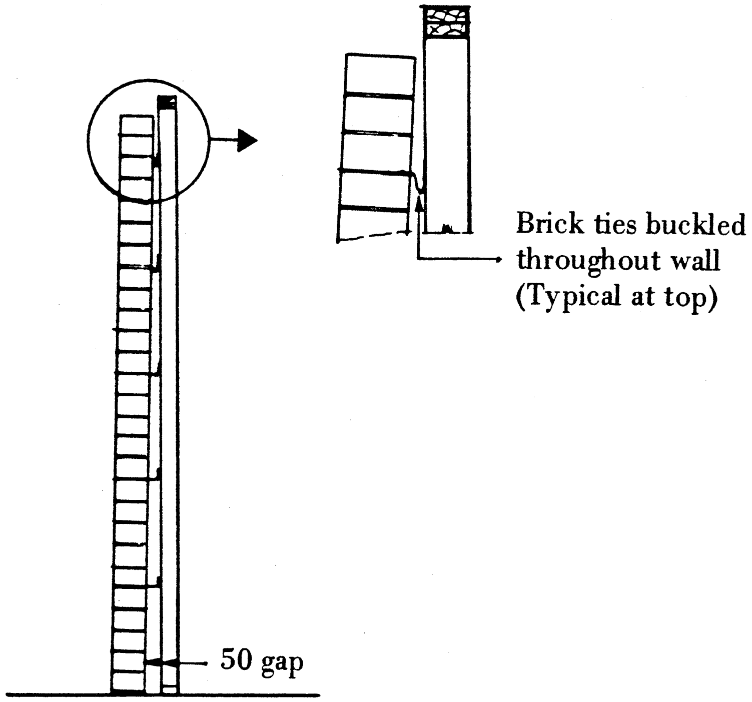


Figure 12 Brickwork failure by Rigid Body Rotation

6. LATERAL AND UPLIFT PRESSURES

This test can be considered the most important of the series insomuch as it stresses the whole of the house frame by applying lateral racking forces to the windward wall simultaneously with uplift pressures to roof structure. As the house had been designed for cyclone prone areas it was loaded with a cyclic loading regime to simulate the extended duration of gustiness within a tropical cyclone. The walls were loaded with the sequence of loading outlined in Section 5.1, that is 1020 cycles building up to design load. The roof structure was also loaded to the provisions of EBS Technical Record 440, but in this case the total number of cycles specified is 10200. This greater number of cycles acknowledges the turbulence at roof level caused by the building itself. Thus the total number of cycles to be applied were as follows

8000 cycles	0 - 5/8 design uplift pressure - 0
2000 cycles	0 - 3/4 design uplift pressure - 0
200 cycles	0 - design uplift pressure - 0

together with

800 cycles	0 - 5/8 design lateral pressure - 0
200 cycles	0 - 3/4 design lateral pressure - 0
20 cycles	0 - design lateral pressure - 0

The equivalent design pressures used for test were 1.23 kPa uplift and 1.01 kPa lateral. Allowance was made for the mass of the loading gear when applying the uplift pressures.

The load cycling operation had been designed to be computer controlled and thus it could be left to run unattended. The computer was programmed to apply nine cycles of uplift load only followed by a cycle of combined uplift and lateral pressure, thereby applying the loading sequences over the same period of time.

Displacement measurements were made using a logarithmic sampling basis, taking readings frequently during the early stages of the test and less frequently towards the end of the loading sequence. A total of 60 displacement gauges were used, three at each end of each alternate truss. Figure 11 shows the positions of the gauges, measuring vertical movement of the truss and vertical and lateral movement of the top plate. It was not practicable to try to measure midspan displacements. For convenience the 19 trusses were numbered from the west end of the house and their ends were designated either north or south. The windward face was therefore the north face. This notation will be used throughout the report. Figure 13 shows the approximate location of the trusses.

6.1 Cycling to 5/8 Design Pressure

The maximum displacement measured during the first load cycle was approximately 5 mm uplift at the north end of truss 7. By about 130 uplift cycles this movement increased to 12 mm but then settled down. It was presumably caused by

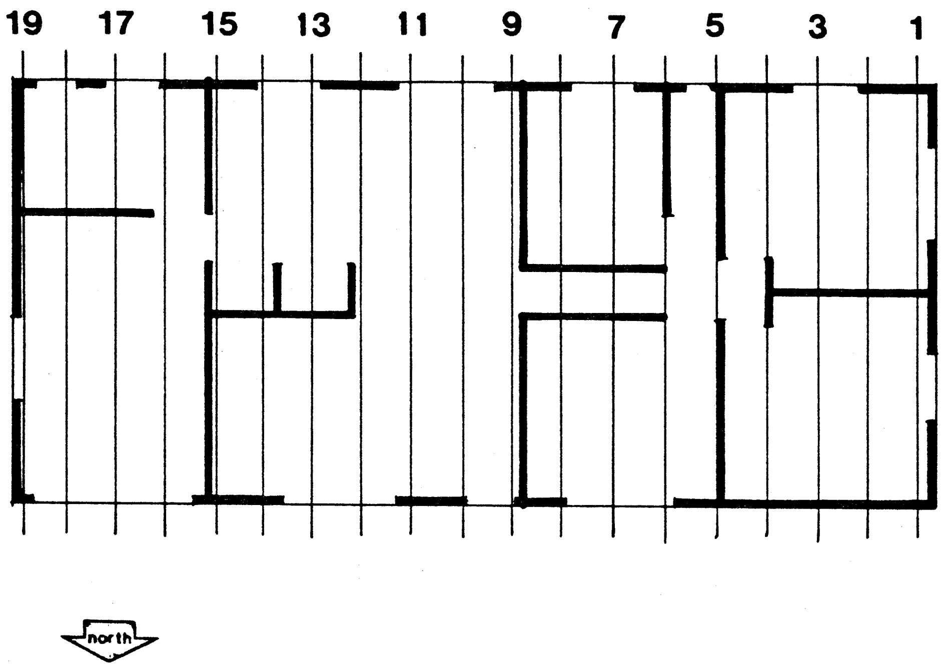


Figure 13 Truss Numbering System

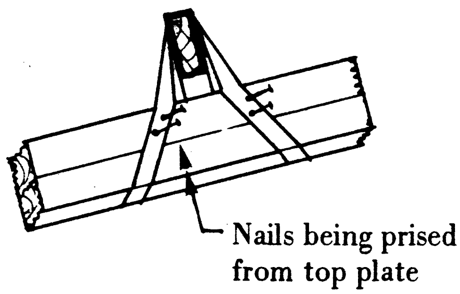


Figure 14 Failure by Nail Prising

the hold-down strap not being very tight. No other trusses exhibited such movement at that early stage of the loading sequence.

After about 2500 cycles of vertical loading and 250 cycles of lateral loading trusses 13, 14, 17 and 19 showed some increase in vertical deflection at their south end. The trusses were lifting up to 8 mm as a result of their having prised some of the 30 x 2.8 mm nails out of the outside face of the top plate. Figure 14 shows the situation at approximately 2500 cycles. One nail had been prised out of two truss straps and other nails were part way out. The test was continued as it was obvious that the straps were still held secure by the nails in the underside of the top plate and on the inside face of the top plate. The hold-down system did not appear to be weakened by the loss of those nails.

At about 3800 cycles of uplift and 380 lateral cycles there was considerable movement at the south east corner of the roof. Trusses 17, 18 and 19 were lifting 15-22 mm. By 4000 cycles the south end of truss 18 lifted completely causing the strap on truss 19 to break. The test was stopped and the situation assessed.

The south end of truss 18 was located on the lintel beam over the laundry doorway. As such, the specified looped strap tie down connection had not been used but had been replaced with a simple strap connection, having straight legs as shown in Figure 15. This type of connection is not in accordance with the provisions of the Home Building Code and therefore should not have been used. At other positions where trusses were located on lintels, two framing anchors were used as tie down in accordance with the code.

The tie down failure at truss 18 was caused by the same prising mechanism previously mentioned for nearby trusses. However because the strap was not looped under the top plate the loss of some nails by prising was critical and resulted in overloading of the remainder. It is assumed that this failure of the tie down at truss 18 caused the fracture of the tie down strap at truss 19.

The two tie down systems were then repaired and the test programme was continued.

After about 5500 cycles of 5/8 times design uplift pressure and 550 cycles of 5/8 times design lateral pressure a close inspection was made of the truss tie

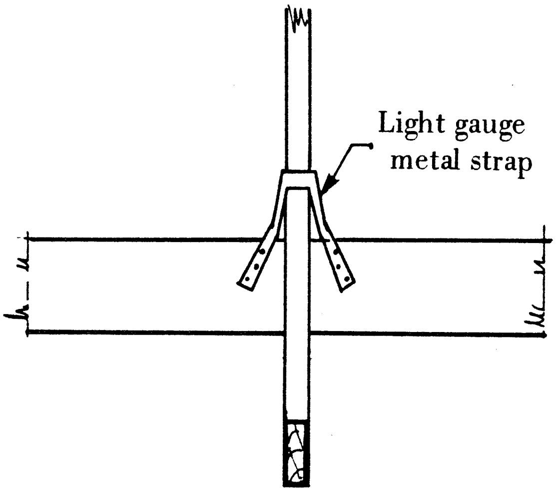


Figure 15 Truss Tie-down over Lintel

down details. Apart from those already repaired, four straps were broken, two were cracked and eleven were prising nails. On the north side the ties at trusses 1 and 3 were broken, at 4 was cracking and at 5, 9, 11, 12 and 13 were prising nails. On the south side the ties at trusses 13 and 16 were broken, 14 was cracked and at 8, 9, 10, 11, 12 and 17 were prising nails. Despite these failures the house was still able to resist the applied loading. At about 6800 cycles of uplift loading the two cracked ties had broken.

The house eventually resisted the full complement of 8000 cycles to 5/8 times design uplift pressure and 800 cycles of 5/8 design lateral pressure. In achieving this, 9 of the 35 truss tie down straps broke and a further 17 prised some nails out of the top plate. There was no concern at all about lateral displacement, as the maximum value recorded by a horizontal gauge was approximately 3 mm.

One noticeable aspect of the test was the performance of the framing anchors securing trusses 16, 17 and 18 over the garage lintel. They showed no sign of distress nor any inkling of prising the nails out of the timber. While they represent only 3 of the total of 38 truss tie downs they do suggest a potentially better performance.

The main concern with metal strap as tie down appears to be the difficulty in ensuring even tightness in each leg of the strap. If the legs of the strap do not have the same tightness or are at odd angles, the force will be different in each. One leg therefore will be overloaded. The roof and ceiling diaphragms would prevent the strap from self-aligning to balance the forces so it is likely that any tendency towards imbalance will remain in the strap.

The preformed shape of framing anchors means that they do not have to be tightened on the job and thus they are less likely to have an imbalance of forces under load.

It was decided to discontinue the schedule of cyclic loading of the house in case the apparently imminent failure of other tie down straps caused unnecessary damage. The straps could not be considered to be a satisfactory tie down medium. It was further decided to remove all the tie down straps, replace them with framing anchors and repeat the cyclic loading schedule. This would give a direct comparison between the likely performance of the two methods.

6.2 Cycling with Framing Anchors

The looped metal straps were replaced with a pair of framing anchors, each having four nails into both the truss and the top plate as specified in Figure 41.9(2)-2 of the Home Building Code. The loading schedule of 8000 cycles of 5/8 times design uplift pressure together with 800 cycles of 5/8 times design lateral pressure was commenced.

The early stages of the test indicated more movement of the top plate than previously. This reflected the fact that the framing anchors could be nailed to the top half of the ribbon plate only.

After 2150 cycles of uplift and the appropriate number of lateral load cycles the computer control system malfunctioned while unattended. The house was severely overloaded in uplift resulting in trusses being torn off the wall on the southern side and roof battens pulling out and breaking on the north side. The ribbon top plate on the south wall fractured in a number of places and was torn off the studs.

After much deliberation and very detailed inspection it was decided to have the house frame repaired and use framing anchors as truss tie down.

6.3 Cycling the Repaired House

Chronologically these cyclic load tests were conducted after the racking series described in Section 7, but logically they should be discussed in this section.

Although some modifications were made to the house during the racking tests their results indicate that there was little effect on the response of the house. It has therefore been assumed that the structural response measured by these tests would be virtually the same as if the tests were conducted immediately after the house was repaired.

Once again the first cycles of 5/8 times design uplift pressure combined with 5/8 times design lateral pressure caused a significant vertical displacement at the ends of some trusses. The north end of truss 13 registered approximately 8 mm vertical movement, 5 mm of which related to movement of the ribbon top plate. The reason for this movement of the top plate has already been discussed in Section 6.2.

By 1380 uplift cycles and the appropriate number of lateral cycles, the gauge at truss 13 north was still registering almost 8 mm per cycle 5 mm of which was movement of the top plate, but nearby trusses were registering similar amounts. Truss 11 north had a net vertical displacement of 7 1/2 mm and truss 15 north had 6 mm. Truss 17 north and the top ribbon were both moving about 6 mm. The south side of these trusses was moving 3-4 mm vertically. There was very little lateral movement of the top plates.

At about 4500 uplift cycles a crack was observed in one framing anchor at the north end of truss 14. The associated deflection was approximately 8 mm. By 5600 cycles this deflection was approximately 10 mm. An overall assessment showed that the top plate near the north end of truss 5 was lifting about 10 mm and that at least one nail was being prised out of the timber by the framing anchor at the north end of trusses 5, 8, 11, 14, 16 and 19 and at the south end of truss 1.

By 7000 uplift cycles the framing anchor on truss 14 north had cracked further and truss 13 was lifting about 8-10 mm and separating the ribbon top plate. The upper member of the top plate had been joined adjacent to truss 13. These increasing movements of the house frame caused the rate of load cycling to decrease significantly. The last thousand cycles in the sequence had an average rate of about 50 seconds.

At about 7500 uplift cycles the ceiling at the midspan of the trusses in the lounge was rising about 40 mm during each cycle. This reflected large displacements at the north end of trusses 12, 13 and 14 which had broken their tie down details. On trusses 11 and 12 both framing anchors had cracked right through, as shown in Figure 16. On truss 14 one framing anchor cracked through and the other pulled out of the top plate. The framing anchors on truss 13 had not broken but had lifted the top section of the ribbon top plate, as shown in Figure 17.

On completion of the 8000 cycles of 5/8 times design uplift pressure and 800 cycles of 5/8 times design lateral pressure an assessment of the damage was made. It revealed the breaking of at least one framing anchor at the north end of four trusses, numbers 8, 11, 12 and 14. Five trusses, 1, 3, 15, 16 and 19, had nails prised partly out of the north end framing anchor. Two trusses, 6 and 13 north, were lifting the top section of the ribbon plate. The only damage on the south side of the house consisted of one nail being prised from the plate at trusses 1 and 19.

Once again it was decided not to continue with the rest of the load cycling regime. It would be most likely that the house would be damaged further if the cycling was continued and that may prevent the rest of the test programme being completed.

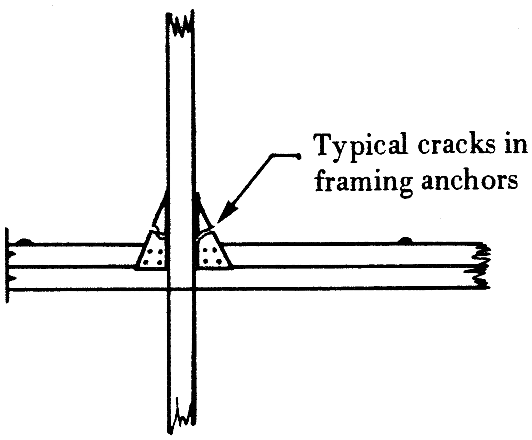


Figure 16 Framing Anchors Cracked

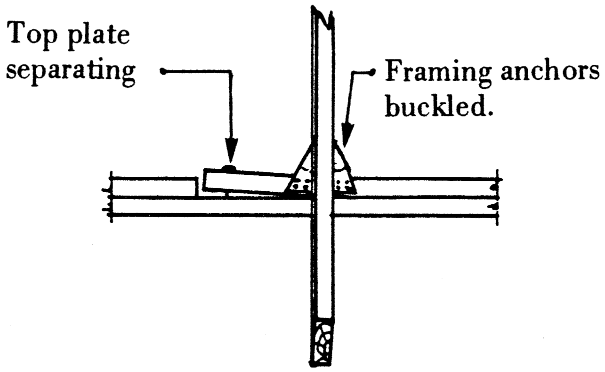


Figure 17 Top Plate Failure

6.4 Assessment of Cyclic Load Tests

Light gauge metal straps were specifically used in the construction of this brick veneer house because of their poor performance in the testing of the Tongan house (Boughton and Reardon, 1984). Their failure to satisfy the full complement of prescribed load cycles in this test series raises further questions about their suitability for use in cyclone prone areas. It is believed that their inclusion in the Home Building Code was based on a series of static load tests which derived a suitable working load for wind gust conditions but not for tropical cyclones. In the light of results of this brick veneer house it is suggested that light gauge metal straps not be used for truss tie down in areas prone to fully developed tropical cyclones.

The framing anchors performed somewhat better than did the light gauge metal straps but still could not be considered satisfactory to resist the prescribed load cycling regime. It is suggested therefore that further investigations be made into the effect of load cycling on this type of connector. In the first instance cyclic laboratory tests could be conducted on individual joints and these could be complemented later with similar tests on structures.

The third aspect that may need to be examined is the accuracy of the simple load cycling regime that has been accepted as standard for simulating the gustiness within a tropical cyclone. It is readily acknowledged that the recipe used is a very simplified version of the complex gust loading caused during a cyclone. It is probable that conservative approximations were made in the simplification and possibly these should be reviewed. The research currently being conducted at James Cook University by Jancauskas, Walker and Doull includes a reassessment of the wind field within a tropical cyclone. It will probably lead to a review of the simplified criteria.

7. EXPLORATORY WALL RACKING

As already mentioned these racking tests were conducted immediately after the house had been repaired, so all of the truss tie down joints were intact.

The aim of these tests was to investigate the effects of systematically disabling the bracing walls within the house. The reason for this was to determine if the current restriction on bracing wall spacing (6 m maximum for cyclone areas) could be relaxed somewhat.

The test pattern was to simulate the racking forces by applying a uniform lateral load to the top plate of the windward wall. The response of the house was measured by 39 displacement transducers located mainly along the top plate of each of the long walls. They were positioned to measure the response of the transverse walls and of the top plate midway between transverse walls. From previous tests (Section 4) 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 to three times design pressure in the first instance and reduce the load for later tests if necessary. For most tests uplift pressure was not applied in conjunction with the lateral forces.

7.1 Disabling Diagonal Bracing

The location of the internal transverse walls braced with diagonal light gauge steel has already been indicated, but Figure 18 shows the position of those walls again together with the location of the horizontal displacement gauges that showed the most movement.

The first test in this series was virtually a repeat of the test described in Section 4.3, but to higher loads. It was conducted as a datum against which the other tests can be gauged. Thus the horizontal force applied to the top plate of the windward wall was incremented uniformly to a total of 60 kN. At this load the maximum deformation was 3.6 mm measured at positions 2 and 38 with 3.5 mm measured at position 10. The other gauges registered less than 3 mm. These displacements are very low considering the applied force was three times the lateral design pressure.

7.1.1 Bracing wall 3

The diagonal braces in wall 3 were then disabled by drilling holes in the plasterboard and fibre cement linings and cutting the braces between each stud and between the appropriate stud and the top or bottom plate. Figure 19 illustrates the effect. The connection between the top of the bracing wall and the roof trusses, as shown in Figure 3, was also dismantled. This alteration meant that there was no specified bracing wall between bracing walls 2 and 4 which were just more than 9 metres apart.

The new house configuration was then tested by incrementing the uniform horizontal load to a total of 60 kN. At this load gauge 2 registered 2.6 mm, gauge 42 read 2.2 mm and gauge 38 indicated 2.8 mm. All other gauges

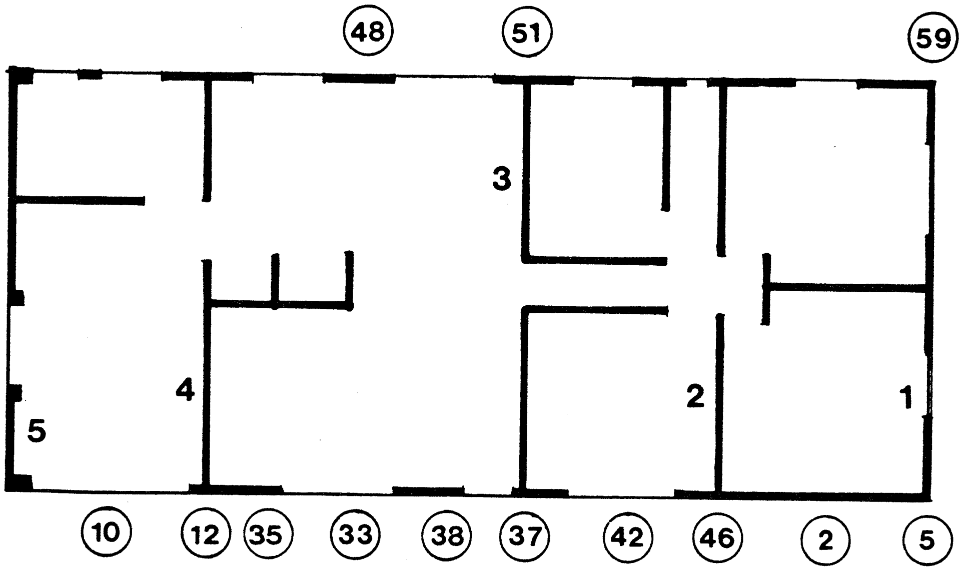


Figure 18 Location of Gauges Measuring Horizontal Displacement

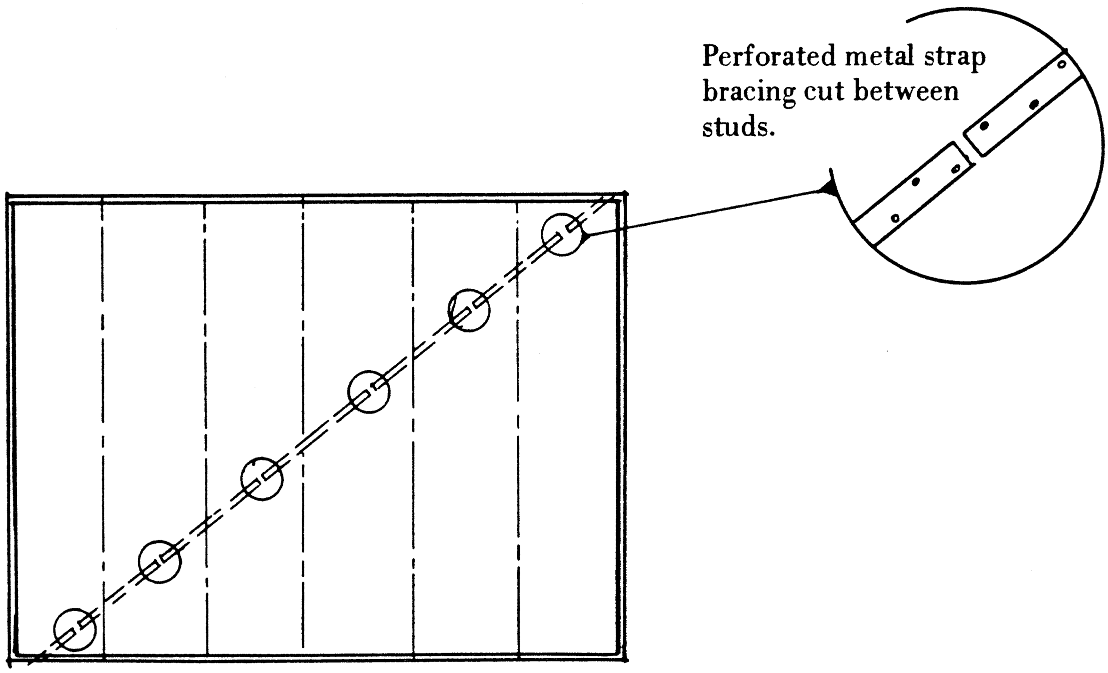


Figure 19 Method of Disabling Bracing Walls

registered less than 2 mm. This overall response of the house indicated that it was stiffer without bracing wall 3 than with that wall. Of course this is a most unlikely situation. The outcome of less displacement probably reflects the usual response of a nailed timber structure by "settling in". This process of settling in means that the timber frame absorbs all tolerances and any other non-elastic deformation in the first load cycle and therefore appears to be stiffer on subsequent loadings. The main result from this test is the indication that the house is not noticeably weaker or more flexible because the braces were disabled.

7.1.2 Longitudinal bracing walls

The theory of shear forces and bracing walls is somewhat complex in that it involves complementary forces acting normal to imposed shear forces. McDowall (1984) showed that bracing diaphragms positioned at right angles to the line of action of the racking forces could contribute significantly to the overall racking response of a building. For this reason the longitudinal bracing sets were disabled in a similar manner to the diagonals in bracing wall 3, as was the connection at the top of the walls. There were two sets, one in the wall between bedroom 2 and the passage and the other in the short longitudinal wall dividing the kitchen and the lounge. As a result of that action the only specified internal bracing left in the house was in walls 2 and 4, but the plywood bracing panels were still in the external walls.

Once again the house was loaded incrementally to a uniform total horizontal force of 60 kN, three times lateral design load. The maximum lateral displacement was measured by gauge 38 as 2.8 mm, gauge 2 measured 2.7 mm and gauge 42 measured 2.3 mm. The other gauges recorded maximum displacements of less than 2 mm. This response was almost exactly the same as that of the previous test, indicating two factors. Firstly, that in this case the flexible diagonal bracing system was ineffective in the complementary bracing mode. Secondly, that the assumption about the house frame settling in during the first test was correct.

7.1.3 Bracing walls 2 and 4

The diagonal braces in wall 2 were cut and the ceiling connections were disabled. The house was retested to 60 kN total horizontal load with almost no change. The maximum horizontal displacements at gauges 38, 2 and 42 were 2.7 mm, 2.5 mm and 1.9 mm respectively.

All the specified bracing in wall 4 was also disabled and the test to 60 kN was repeated. The maximum horizontal displacements at gauges 38, 2 and 42 were 2.7 mm, 2.6 mm and 2.2 mm respectively.

7.1.4 Tie down nullified

Each of the internal walls that had braces incorporated were secured to the concrete floor slab by a 10 mm diameter bolt at each end. These bolts provided resistance against both sliding and overturning. It was decided to remove the nuts from all the bolts and thus remove the restriction against overturning while still preventing sliding. Actually the bolt may have still offered some resistance to overturning by friction, especially at the thread, but it was considered too drastic to saw the bolts off.

The house was retested to 60 kN total horizontal force. It did respond to this latest modification, with a maximum displacement of 3.5 mm at gauge 38 and displacements of 3.3, 3.3, 3.1, 3.0, 2.9, 2.4 and 2.3 mm at gauges 42, 2, 33, 10, 35, 12 and 37 respectively. All other displacements were less than 2 mm. The recorded displacements reflect the greater movements at walls 3 and 4 indicated by gauges 37 and 12. However, put into perspective the maximum displacement is still only about 3 mm at three times design lateral pressure.

7.1.5 Summary

Table 2 lists a summary of the response of the house at various locations to the applied racking force of 60 kN. Each test configuration includes the accumulation of effects of the previous tests, that is, there were no repairs made after each test. Where displacements are not included in the table, they were very small by comparison. The location of gauges is shown in Figure 18.

The results of these lateral load tests show that the test house, with its plasterboard and fibre cement board internal wall lining and its plasterboard ceiling, did not really need the diagonal bracing sets for its racking strength and stiffness. The disabling of those diagonal braces had little effect on the house performance at elevated loads. The internal linings, although only glued or nailed to the manufacturer's specification for partition walls or ceiling, had sufficient strength and stiffness to either act as bracing media or to transfer the forces in diaphragm action to the external bracing walls.

TABLE 2

RESPONSE OF HOUSE TO 60 kN LATERAL LOAD

Test	Member Nullified	Displacement (mm) at gauges 10	12	35	33	38	37	42	2
1	None	3.5				3.6			3.6
2	Diag. braces, wall 3					2.8		2.2	2.6
3	Long. Diag. braces					2.8		2.3	2.7
4	Diag. braces, wall 2					2.7		1.9	2.5
5	Diag. braces, wall 4					2.7		2.2	2.6
6	Hold-down	3.0	2.4	2.9	3.1	3.5	2.3	3.3	3.3

The fact that there was little change in response of the house means that the internal partitions were acting in bracing even before the diagonals were cut. This is not surprising as the glued lining boards would offer a much stiffer (and therefore preferred) line of action to any lateral forces. In assessing the situation further, there is a total of 40 lineal metres of lining board on the main internal walls to resist a total of 20 kN uniform design racking force. At an average force of 1/2 kN per lineal metre of board (or 1 kN per metre of wall) the internal lining can easily cope with the lateral loads, and even the test load of 1 1/2 kN/m of board.

It must be stressed however that these results were achieved with relatively new lining board, with young glue and with the boards fixed strictly according to the manufacturers' specifications and, in the case of the plasterboard, in the presence of his representative. The results may not quite be so favourable under different circumstances. Also, from the practical point of view, diagonal or some form of bracing is necessary to keep the wall frames square during construction.

7.2 Disabling Internal Walls

Having investigated the structural response of the house as the designated braces were sequentially removed, the next obvious step was to investigate its performance as the wall lining was removed. This was effectively equivalent to removing the walls, as the unbraced timber wall frame would offer no racking resistance. The walls chosen for this series of tests were bracing wall 3 and its extension dividing bedroom 2 and the lounge. Removal of the latter wall would simulate a large lounge room approximately 9 m long, and removal of the remainder of bracing wall 3 would leave a very large open area.

In removing the lining from each face, care was taken not to disturb the cornice and to leave a 50 mm wide strip of lining at the top of each wall. This was meant to simulate continuity of the ceiling for the situation where the wall was not there.

7.2.1 Lounge/bedroom wall

For the first test in this series the plasterboard lining on the lounge side of the wall was carefully removed, leaving only the 50 mm strip below the cornice. The load was incremented to 60 kN and the lateral response of the house was recorded. It was little different from that measured in previous tests, gauge 38 recorded 3.6 mm with gauges 33, 2, 10 and 42 recording 3.4, 3.2, 2.8 and 2.3 mm respectively. The other gauges measured less.

As that test indicated virtually no change in response of the house cladding on the other face of the wall was removed. The condition then simulated a 9 m space between the lounge/garage wall and the bedroom 1/bedroom 2 wall, representing a very large lounge room. Once again the load was incremented to 60 kN and the response of the house changed very little. Gauges 38, 2, 33 and 42 recorded 3.6, 3.4, 3.3 and 3.0 mm respectively.

7.2.2 Kitchen/bathroom wall

This wall, which was bracing wall 3 in its original configuration, had the plasterboard carefully removed from the kitchen face first. The test to 60 kN uniform load showed virtually no change in response of the house.

The fibre cement lining was then removed from the wall leaving only the unbraced studs remaining as what had been bracing wall 3, and a similar wall of unbraced unclad studs separating the lounge and bedroom. Once again the house was loaded to three times its lateral design load but on this occasion it did register a change in response. Gauge 37, which had been measuring the racking deflection in line with walls being disabled, showed a significant increase in response and registered 4.1 mm. This led to an overall increase in other readings recorded in the vicinity of the lounge. Gauge 38 recorded 5.3 mm and gauges 33, 2, 35 and 12 recorded 4.1, 3.7, 3.2 and 3.0 mm respectively.

To complete the series of tests the plasterboard lining was removed from both faces of both passage walls, in case there was any contribution from them. This effectively left the kitchen, lounge, bathroom and bedroom 2 as one large open space.

The test was repeated and the load incremented slowly to 60 kN. The response to the loading was slightly greater as 9 gauges recorded displacements of 3 mm or more. Gauge 38 still recorded the greatest movement of 5.8 mm. Movements of 5.2, 4.5, 4.3, 4.3, 4.0, 3.8, 3.7 and 3.0 mm were recorded by gauges 33, 42, 35, 37, 2, 10, 12 and 46 respectively.

7.3 Racking and Uplift

In this series of exploratory wall racking tests only the racking forces have been applied, albeit to three times design load. A final test was therefore conducted to investigate the effect of uplift pressure on the roof structure combined with the racking forces. The house configuration was still that described for the last test in Section 7.2.2, that is, with the large open space in place of the kitchen, lounge, bathroom and bedroom 2.

As these tests were conducted before the cyclic loading tests already described in Section 6.2 it was decided not to overload the house in uplift, in case it caused failure of any member. The sequence of loading for this combined load test was therefore to apply the full design uplift pressure before applying the lateral pressure in the usual increments to 3 times lateral design pressure.

Displacement movements were certainly greater than for the equivalent test without uplift, but were still relatively small considering the magnitude of the lateral force. The main property that was monitored closely in this test, as well as in the previous ones, was that the response was elastic. There was certainly no evidence of yielding at the high load applied.

The maximum displacement of 7.6 mm was still recorded by gauge 38. Other displacements were 6.2, 6.0, 5.9, 5.8, 5.6 and 4.8 mm measured by gauges 33, 35, 2, 37, 42 and 10.

7.4 Summary

Table 3 lists a summary of the response of the house to the removal of the internal cladding. Where the displacements were very small they have not been included in the table. The gauge locations are as shown in Figure 18.

TABLE 3

RESPONSE TO REMOVAL OF INTERNAL LINING

Removed from	Displacement (mm) at gauges 10	12	35	33	38	37	42	2
1 face, lounge/bed	2.8			3.4	3.6		2.3	3.2
both faces, lounge/bed				3.3	3.6		3.0	3.4
1 face, kitchen/bath			2.7	2.6	3.5		2.5	3.2
both faces, kit/bath		3.0	3.2	4.1	5.3	4.1		3.7
both faces, passage	3.8	3.7	4.3	5.2	5.8	4.3	4.5	4.0
Racking & uplift	4.8		6.0	6.2	7.6	5.8	5.6	5.9

The results of these tests and the preceding ones indicate that the house had sufficient racking strength and stiffness without the internal walls that were disabled. Given the reservations expressed in Section 7.1.5 about the application of the cladding, the results show that for this type of construction the current regulations limiting the spacing of bracing walls may be conservative when referring to cyclone-prone areas.

It should be noted that the cyclic load tests already described in Section 6.3 were conducted after the racking tests described above and thus some of the internal walls had been nullified.

8. RACKING TESTS TO DESTRUCTION

The final set of tests on the brick veneer house was meant to determine its overall racking strength and, if any of the walls remained intact, the racking strength of individual walls. Of special interest would be the two end brick walls. Although they had been closely monitored throughout the entire test series they had shown virtually no response to the racking forces. This indicated that they were either extremely stiff or that the applied load was not being transferred to them.

The internal layout of the house was still without the walls that had been systematically disabled for the last tests. No repairs had been made.

8.1 Uniform Racking

The uniform racking load was applied to the top plate of the north wall as described in Section 7. Thirty nine displacement gauges were used to monitor the response of the house. They were at the same positions as for the previous racking tests, the location of the principle ones being shown in Figure 18.

The uniform racking load was incremented to 60 kN without much change in response from the previous test under such conditions. The maximum displacement at gauge 38 was 5.6 mm compared with 5.8 mm during the last test. However there was a small change in response of the top plate, as the maximum deflection was 6.3 mm at gauge 42 which measured displacement at the window head in bedroom 2.

At 80 kN the top of the windward wall had displaced 8.7 mm at gauge 42 and 7.8 mm at gauge 38. The effect of the open area was being felt as gauge 37, located where bracing wall 3 had been, registered 7.3 mm and gauge 51 on the leeward face measured 4.4 mm. A thorough inspection was made to determine any evidence of failure, but there were no signs of failure at this load equivalent to 4 times design racking load.

The total load was incremented to 100 kN by which time gauge 42 measured 12.1 mm, gauge 38 measured 11.3 mm and gauges 37 and 35 measured 10 mm. Although there was some audible evidence of strain there were no visible failures.

At 108 kN total load the first visual evidence of failure occurred, a compression failure in the plasterboard wall lining by the kitchen/laundry doorway. It can only be considered a minor failure as it had no effect on the load carrying capacity and it did not cause any immediate increase in deflections.

The total load was increased to 120 kN by which time gauge 42 was measuring almost 17 mm and the other gauges nearby were measuring about 15 mm. Gauge 51 on the leeward face measured almost 11 mm, showing the effect of the 9 m distance between internal walls. A thorough inspection revealed that there was only one extra failure of significance, a crack in the plasterboard at the corner of the window in bedroom 3. The crack would have been caused by the diagonal tension component of the racking force.

Although only the one significant failure was evident, there were many instances of imminent failure. Nail heads had pulled well into the plasterboard at the bottom plates of all internal walls. Where the walls were clad with fibre cement board it was breaking away in small triangles emanating from the nails. Thus, as it was apparent that total failure would probably soon occur, the test was concluded. The 120 kN uniform racking force applied to the windward wall was equivalent to 6 times the lateral design load for the house.

8.2 Brick Veneer Skin

Although the Home Building Code makes no allowance for any contribution to bracing from the brick veneer skin, the Domestic Construction Manual does include some recommendations. Using values from Table 5 in the Technical Notes included in Section D3 of the Manual, a design racking strength of 3.4 kN can be calculated. This estimation is probably somewhat conservative in that it assumes the end wall to comprise three individual panels between windows, making no allowance for continuity above or below the windows. However the interpretation is in accordance with the recommendations given in the Home Building Code for estimating the length of bracing walls.

The top brick at the west end of the windward wall was carefully removed so that the racking equipment could be installed. The horizontal racking force was applied to the top course of veneer brickwork on the west wall. Displacement gauges were positioned to monitor movement of the brickwork and of the timber frame.

At a racking load of 20 kN no discernable movement had occurred. The same situation prevailed after the load had been incremented to 30 kN. At 32 kN a displacement of 0.4 mm was registered and at 34 kN the wall failed. The wall cracked along the mortar joints to the lintel beam over the window in bedroom 1 and then along the mortar joint at the bottom of the window openings. On removal of the applied load the cracks were hardly noticeable.

There was no measurable displacement of the timber frame to which the veneer skin was attached.

8.3 Bracing Wall 1

Bracing wall 1 had a total of 3 m of plywood bracing with a design value of 6.7 kN. During construction the wall had been racked to 10 kN as part of the investigation of roof and ceiling diaphragms.

For this test the roofing and ceiling were left intact so there was provision for the sharing of load to the other walls within the house. As with the previous racking test on wall 1, the concentrated load was applied to the top plate of the windward wall, within 50 mm of the face of wall 1.

The gauges were located in the same positions as for the tests described in Section 7.1, the more important ones being shown on Figure 18. The racking force was incremented slowly and displacements were recorded. At 10 kN the only significant displacement was at gauge 5 which was recording the total movement at the top of wall 1. It recorded 2.6 mm. At 20 kN racking force the displacement at gauge 5 had increased to 5.4 mm. Gauge 59, at the other end of the same wall, measured 1.8 mm. It is obvious that the movement of wall 1 was still elastic.

Wall 1 remained elastic at an applied load of 30 kN when the measured displacement at gauge 5 was 8.0 mm. By this time gauge 59 was measuring 3.3 mm and gauge 46 at wall 2 was measuring 2.1 mm. The response of the wall remained linear until 38 kN when its displacement was 10.2 mm and that of the other end was 4.2 mm. However at 40 kN racking force the displacement measured by gauge 5 increased to 13.4 mm. Failure occurred before the next increment could be applied. The failure was not a racking failure of wall 1, but a shear failure of the top plate of the windward wall to which the load was being applied.

A close inspection of wall 1 showed that the plasterboard had cracked at a corner of the window in bedroom 1 and also that the nail heads were pulling deeply into the plasterboard along the bottom plate. The brick veneer skin prevented an inspection of the plywood in the cavity, the designated bracing medium. As a result of the inspection and the sudden change in load deformation characteristics of the wall it was concluded that the wall would have little reserve strength above the 40 kN applied, which was virtually 6 times its bracing strength.

It should not be concluded however that wall 1 was six times as strong as necessary. A significant portion of the applied load would havae been transferred to the other walls by the diaphragm action of the ceiling.

8.4 Bracing Wall 2

Although bracing wall 2 had a designated strength of only 2.4 kN, the appropriate value for diagonal bracing, the preliminary test programme demonstrated that its actual strength was far in excess of that value. In fact, during these preliminary tests the wall was loaded to 12 kN.

For this final test the ceiling diaphragm was left intact and the load applied to the windward wall top plate adjacent to this bracing wall 2, the brace of which had been disabled during the tests described in Section 7.1.3. Displacement gauges were in the same positions as for the last test, with the more important ones shown on Figure 18. At an applied load of 10 kN gauge 46, which measured the displacement of the wall, recorded only 1.7 mm movement. None of the other gauges recorded more than 1 mm. The load was increased in increments to 20 kN with gauge 46 recording 5.4 mm and gauges 42 and 2 both recording 2.2 mm. By 30 kN those three gauges were recording 9.5 mm, 3.3 mm and 3.4 mm respectively. The load deflection curve for bracing wall 2 was still basically linear at 30 kN.

By 34 kN the load deflection curve was starting to deviate from a straight line and at 38 kN gauge 46 was registering 14.5 mm. At other locations, gauge 2 registered 4.5 mm and gauge 42 registered 4.1 mm. One gauge of interest was 47 which measured displacement of the brickwork at the top course adjacent to the window in bedroom 2. It registered 5 mm at the 38 kN racking load. This movement would be because of failure of the brick skin under the uniform lateral load test described in Section 5.

Once again failure occurred at the top plate where the load was being applied, at a load of 38.4 kN. A close inspection of the wall separating bedrooms 1 and 2 indicated that the nails in the bottom plate were causing local bearing failures in the plasterboard. This distress was not so evident in the wall separating bedroom 3 from the toilet but that wall was probably helped to a large extent by the other toilet wall. Although imminent failure was not quite so obvious from inspection, the decision was made not to retest the wall.

8.5 Bracing Wall 4

Bracing wall 4 was the third of the internal walls that had diagonal bracing specified as the official bracing medium. Thus the specified design racking strength was 2.4 kN before the brace was disabled. Bracing wall 4 was different from wall 2 insomuch as it had one face clad with plasterboard and the other face clad with fibre cement board, in the garage and laundry areas. Also there was no cornice on the fibre cement face but a timber quad had been nailed to the top of the wall and to the ceiling battens.

The aim of this test was to cause the wall to fail in racking, so the load was applied to the large lintel beam over the garage doorway. It was anticipated that the lintel would not fail in shear. The point of application of the load was within 50 mm of bracing wall 4.

The wall was loaded in increments to 10 kN, causing a displacement of only 1.9 mm. The loading was increased in increments to 20 kN by which time the displacement was 5.2 mm. All other gauges were registering movements of less than 2 mm. By 30 kN the displacement of gauge 12 had increased linearly to 8.3 mm and at 36 kN it recorded 10.4 mm and was still linear. The test was then terminated , the load removed and the situation assessed.

The elastic response up to 36 kN was somewhat surprising as this wall had suffered a number of minor racking failures during the uniform load test described in Section 8.1 and was part of the basis for the decision that "lateral failure of the house was imminent". The fact that the response was still elastic indicated that the wall probably still had a significant reserve of strength. It was therefore decided to isolate the wall from the ceiling diaphragm to determine its individual strength. This was achieved by smashing the cornice and removing the timber quad.

The wall was then loaded incrementally to 32 kN when gauge 12 registered 8.5 mm and the response was still elastic. The test had to be terminated because of a loading rig malfunction. As there were no signs of further failure the wall was isolated more by sawing through all of the ceiling battens that were attached to it. The ceiling lining was also broken between the windward wall and the first ceiling batten for the length of the lounge room and the width of the garage, to prevent the applied force from "leaking" into the ceiling diaphragm. Similarly the truss tie down to the windward wall was cut for three trusses on each side of the wall. This left the wall connected to the house frame at only its junctions with windward and leeward walls.

In this virtually isolated state bracing wall 4 was loaded incrementally in racking. At 20 kN the wall had deflected 5.9 mm, that is, approximately half a millimetre greater than for the first test with the ceiling intact. At 30 kN gauge 12 registered 8.1 mm (compared with 8.3 mm in the other test) and at 36 kN the gauge registered 9.1 mm (compared with 10.4mm). At 40 kN the load deflection curve started to show some yielding and at 46 kN gauge 12 registered 11.9 mm. The test had to be terminated at this load as it was virtually the maximum capacity of the lateral loading equipment.

Once again the wall appeared to be on the verge of failure. The nails at the bottom plate had pulled deeply into the plasterboard but were apparently still holding. In the garage the fibre cement lining had buckled near the loading point and there were many cracks emanating from the fasteners, but the wall had withstood the applied load of 46 kN.

In a final effort to cause failure the top plate on the leeward wall was cut on each side of bracing wall 4. This, combined with the previous measures, should have totally isolated the wall. The wall was then loaded incrementally to 40 kN at which time gauge 12 registered 10.4 mm and the load deflection curve remained linear. There was not much movement elsewhere, the greatest value being 3.9 mm at gauge 10. At the next load increment to 42 kN gauge 12 showed a significant increase, indicating that the wall was starting to yield. Failure occurred by racking at 43.6 kN.

The mode of failure was an extension of those failures already mentioned. The fibre cement board tore in triangles from the fasteners to the edge of the sheet. There was a buckling failure near the point of application of the racking load and a stud lifted from the bottom plate at the doorway. The plasterboard on the opposite face also buckled near the point of application of the load causing a glue failure at the adjacent stud. The nails in the bottom plate pulled through the plasterboard.

8.6 Bracing Wall 5

Apparently research into the bracing strength of brick walls was conducted after the advent of the Home Building Code, as that document allows a design value of 4 kN for a single skin wall with a pillar each end located on a concrete slab. The Domestic Construction Manual, published after the Code, has much more liberal values of racking strength. The external wall of the garage therefore had a design racking strength of either 8 kN (from the Code) or 33.7 kN (from the Manual). Although the former value was used in the design of the test house, the racking test already conducted on the brick skin indicates that the latter value may be more realistic. For that reason the loading equipment for racking was strengthened prior to this test on the brick wall.

The horizontal racking force was applied to the reinforced pier at the north end of wall. At 10 kN none of the displacement gauges registered any movement.

The load was incremented in 2 kN steps and at 40 kN there was still no measureable displacement. At 60 kN the top of the pier had displaced only 0.8 mm. This displacement increased to 2.4 mm at 70 kN racking force and to 6.2 mm at 76 kN. The displacement was the result of a local failure of the brickwork pier. It was obvious that the pier was being subjected to torsional stresses as the line of application of the load was eccentric from the line of resistance, the bracing wall. Figure 20 shows the situation.

The load was increased to 80 kN resulting in a displacement of 7.3 mm. For safety reasons the test was terminated at that load. While it was not anticipated that the wall would fail in racking there was concern about the torsional cracks. The pier was cracked about 5 courses below its top and the crack extended for about a metre into the single leaf bracing wall.

Although bracing wall 5 did not fail in racking, the torsional failure of the pier must be accepted as failure of the system. The test load was applied across the full width of the pier, thus its centroid was on line with that of the pier. In practice the wind load could be even more eccentric, especially if the garage door slid in guides attached to the side of the pier.

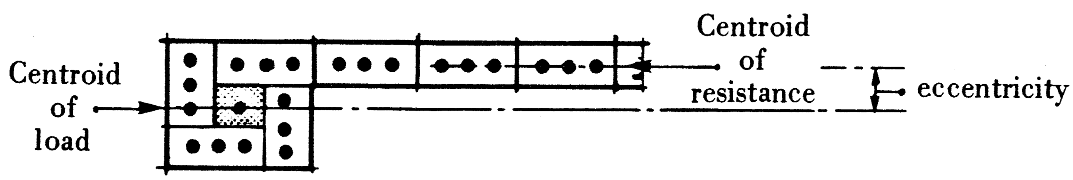


Figure 20 Forces Causing Torsion on Brick Pier

9. CONCLUSIONS

The results of the tests on this brick veneer house are very interesting in that they have highlighted both inherent strength and inherent weakness in the house that was built according to the provisions of the Home Building Code. The extremely high racking strength of the house demonstrates the contribution made by the internal lining board, both as wall bracing and as ceiling diaphragm. The tests demonstrated most aptly that the main function of diagonal bracing is to keep the frame square during construction. They also demonstrated that for this type of construction the maximum spacing of bracing walls could be increased to 9 m. Although the designated braces in those walls 9 m apart had been nullified, they should still be considered as bracing walls.

The poor performance of the looped metal strap under cyclic loading verified the results obtained in previous house testing. The framing anchors performed better than the looped strap but were still not able to sustain the full complement of cyclic loading. It is recommended therefore that further investigations be made into both the fatigue characteristics of light gauge metal and the suitability of the cyclic loading regime in simulating the effects of a tropical cyclone.

The brickwork performed satisfactorily in resisting lateral forces and had high racking strength, although it may be difficult in brick veneer construction to fully utilize that strength as there was no evidence of the framework and the veneer interacting as bracing.

A summary of the main conclusions is listed below:

(i) The house as originally constructed with light metal straps as truss tie down elements was unable to resist the prescribed cyclic loading regime. The test was terminated after application of 8000 cycles to 5/8 times design uplift pressure and 800 cycles to 5/8 times design lateral pressure.

(ii) On termination of the cyclic loading test, 9 of the 35 tie down straps had broken and 17 more had prised some nails out of the top plate.

(iii) Framing anchors performed better than looped straps as truss tie down elements but were also unable to resist the full complement of cyclic loading. The test was terminated after application of 8000 cycles to 5/8 times design lateral pressure.

(iv) On termination of the cyclic loading test framing anchors had broken at 4 of the 38 truss tie down locations and 7 others were prising nails.

(v) The house was very strong in racking as it was able to withstand in excess of 6 times its design racking force.

(vi) The internal lining, although not fixed to the standards required for bracing walls, provided most of the racking strength.

(vii) The diagonal braces that were designated as bracing elements were ineffective because of the strength and stiffness of the internal lining. Their removal did not affect the response of the house.

(viii) After the removal of some internal walls, resulting in a virtual 9 m spacing between bracing walls, the house withstood 3 times design racking pressure combined with once design uplift pressure without serious distress.

(ix) Initial tests showed that both the metal tile roofing and the ceiling acted as diaphragms. The flat plasterboard ceiling was by far more efficient.

(x) The internal non-bracing walls were probably able to withstand well in excess of the applied racking forces of 38 or 40 kN, with the ceiling still attached.

(xi) The dividing wall between lounge and garage and its extension dividing the kitchen and laundry, after having its diagonal bracing cut and having been isolated from the rest of the house, failed in racking at a force of 43.6 kN. This is equivalent to a force of 7.8 kN per metre.

(xii) The brick veneer skin showed little response to the application of 1020 cycles of lateral pressure building up to design load.

(xiii) The brick veneer skin eventually failed as a rigid body rotating about its base as the brick ties buckled. The failure occurred at a uniform pressure equivalent to 2.95 times the design pressure.

(xiv) The racking strength of the brick veneer skin was quite high, needing a force of 34 kN to cause failure. This is equivalent to 4.7 kN per metre over the full length of the wall that was tested.

(xv) There was no interaction between the brick skin and the timber framed wall to which it was tied when either element was subjected to racking forces. The brick ties were far too flexible to transmit racking forces.

(xvi) The single leaf brick wall of the garage, with its engaged piers, was extremely stiff and strong in racking. The eventual failure of the wall at a load of 80 kN was by torsion of the loaded pier rather than by racking. This is considered a valid failure mode for the wall.

(xvii) The use of framing anchors with ribbon plate construction is not recommended as the anchors can be attached to the top laminate only.

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11. REFERENCES

BEST, R.J. and HOLMES, J.D. (1978). "Model Study of Wind Pressures on an Isolated Single Storey House". Wind Engineering Report 3/78, James Cook University of North Queensland.

BOUGHTON, G.N. (1983). "Load and Deflection Instrumentation for Structural Testing". Technical Report No. 18, James Cook Cyclone Structural Testing Station.

BOUGHTON, G.N. (-), "An Investigation of the Behaviour of Full Scale Houses under Simulated Wind Loads". Ph.D. Thesis (in preparation).

BOUGHTON, G.N. and REARDON, G.F. (1983). "Testing a High Set House Designed for 42 m/s Winds". Technical Report No. 19, James Cook Cyclone Structural Testing Station.

BOUGHTON, G.N. and REARDON, G.F. (1984). "Simulated Wind Loads on the Tongan Hurricane House". Technical Report No. 23, James Cook Cyclone Structural Testing Station.

EXPERIMENTAL BUILDING STATION (1978). "Guidelines for the Testing and Evaluation of Products for Cyclone Prone Areas", Technical Record 440, Department of Housing and Construction.

HOME BUILDING CODE QUEENSLAND (1982). Appendix 4 to the Standard Building By-Laws 1975, Queensland Government Printer.

JANCAUSKAS, E.D., WALKER, G.R. and DOULL, G.R. (1978). "The Fatigue Characteristics of Wind Loads on Low Rise Buildings", Paper submitted to First National Structural Engineering Conference 1987.

McDOWALL, G.C. (1984). "Qualitative Assessment of Whole House Structural Response to Lateral Wind Loads". Proc. Pacific Timber Engineering Conference.

REARDON, G.F. (1985). "Structural Testing of Houses for High Winds", Queensland Division Technical Papers, Institution of Engineers, Australia.

REARDON, G.F. and BOUGHTON, G.N. (1984). "Testing a Logan Unit House Designed for 63 m/s Winds". Technical Report No. 22, James Cook Cyclone Structural Testing Station.

ROY, R.J. and WALKER, G.R. (1984). "Total Load Measurements on Low Rise Buildings: Terrain Category 2". Proc. Workshop on Wind Engineering and Industrial Aerodynamics.

SMITH, P.R. and ADAMS, R.H.T. (1984). "Domestic Construction Manual". Queensland Master Builders' Association.

STANDARDS ASSOCIATION OF AUSTRALIA (1979). SAA Timber Framing Code AS1684-1979.

STANDARDS ASSOCIATION OF AUSTRALIA (1983). "SAA Loading Code - Wind Forces", AS1170, Part 2-1983.

TRADAC (1985). "Queensland Timber Framing Manual - W42". Timber Research and Development Advisory Council.