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TESTING A LOGAN UNIT HOUSE DESIGNED FOR 63m/s WINDS

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G.F. REARDON

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TESTING A LOGAN UNIT HOUSE
DESIGNED FOR 63 m/s WINDS

G.F. Reardon*
G.N. Boughton**

SUMMARY

Simulated wind load tests were conducted on a panelized Logan Unit house which had been engineered to withstand cyclonic winds in a very exposed location. Test loads were determined by comparing the results from a model in the wind tunnel with those calculated according to the Wind Loading Code. A study was made of the diaphragm action of the roofing and ceiling panels as well as the bracing capacity of the wall panels, both during construction of the house and during the test programme.

The cyclic loading sequence, simulating the buffeting action of cyclone winds, highlighted the need to redesign a bracket over a window head. The redesign was effected and the modified house was retested. Eventual failure occurred at an uplift force equivalent to twice design load, combined with a lateral force in excess of twice design load. The house proved to be very stiff and strong in resisting lateral loads.

* Technical Director, Cyclone Testing Station
** Mount Isa Mines Research Fellow, Cyclone Testing Station.

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1. INTRODUCTION

It is readily acknowledged that the housing industry cannot yet afford to have individual houses engineered to resist all the loads they will probably encounter during their lifetime. State building regulations are intended to provide a minimum standard to which buildings can be built, and they sometimes emphasize the hazards most likely to occur within the state. In this context the Queensland Home Building Code (1981) emphasizes building techniques to resist high winds and tropical cyclones.

If a common set of building components are used for a number of different houses, these components can afford a greater engineering input than individual houses. This is one of the efficiencies of prefabricated construction. Individual elements can be tailored for their exact role within the structure, and the size of the building can be changed by adding or subtracting elements.

The Logan Unit building system works on this principle. The basic element is a light gauge metal frame clad on each face with fibre cement sheeting. The frames are anchored to concrete slab and to each other. As all Logan Unit houses are built with these panels, a considerable amount of engineering can be used to make them efficient and versatile.

Despite their apparently simple construction most house building systems pose a very complex structural analysis problem and their ultimate strength may be quite different from that predicted by simple structural analysis. This was emphasized in tests conducted on a timber framed house (Boughton and Reardon, 1983), where some parts of the structure were much stronger than necessary and other parts were not strong enough.

When tests on a well designed building show it to be stronger than anticipated it is usually because of a redistribution of load within the structure. This occurs when excessive deflection, or even partial failure, of one element allows it to be supported by adjacent stiffer or stronger elements. In this case some of the applied force is distributed to the adjacent elements and the weaker one does not fail completely. Such a mechanism is difficult to allow for in structural design or laboratory tests of individual elements.

If the tests show up a weakness, experience has shown that it is usually associated with a small detail that can easily be overlooked during the design process.

It was for these reasons that Logan Units commissioned the Cyclone Testing Station to test their "Carnarvon" style house, designed to be erected in a very exposed location within the proclaimed cyclone zone. This floor plan was chosen because of its relatively open living area, which was expected to be the critical part of the design.

2. DESCRIPTION OF THE HOUSE

2.1 The Logan Unit System

As with other prefabricated building systems, the basic concept of the Logan system is to produce house components in a factory and keep on-site labour to a minimum. The advantage of this is that the factory produced components are identical and are very easily assembled. The reduction in on-site labour is meant to keep delays to a minimum. The system is also aimed at the owner-builder market.

The concrete floor slab must be constructed accurately and the perimeter holding down bolts installed in position whilst the concrete is wet. When the slab has cured the position of internal walls is marked and masonry anchors installed at module intervals. The wall panels are then bolted to anchor plates secured by the holding down bolts. Adjacent panels are bolted together at the top, through a bracket that also holds down the roof structure. Thus the roof principals are spaced at module intervals. Cover strips between adjacent wall panels provide a weather seal, and skirting boards hide the ironmongery. Figure 1 illustrates the principle.

A standard modular wall panel is 1016 mm wide, 2438 mm high and 76 mm thick. Its components are channel shaped light gauge steel and 6 mm fibre cement sheeting.* There is a stud at each end and a top, bottom and three intermediate rails. Tabs on the rails fit through slots in the studs and are bent over to keep the rails in position. There are no diagonal braces. The sheeting is glued to each face of the steel frame and held in position by three screws per stud. External or internal grade sheeting is used as required. Polystyrene batts 19 mm thick are installed in the panels during fabrication. Other panels are supplied with window frames installed, or as door frames with pre-hung doors.

**A spot check showed that at least some sheets were asbestos cement, which is stronger than fibre cement sheeting. The ramifications of this are discussed in Section 6.4.3.*

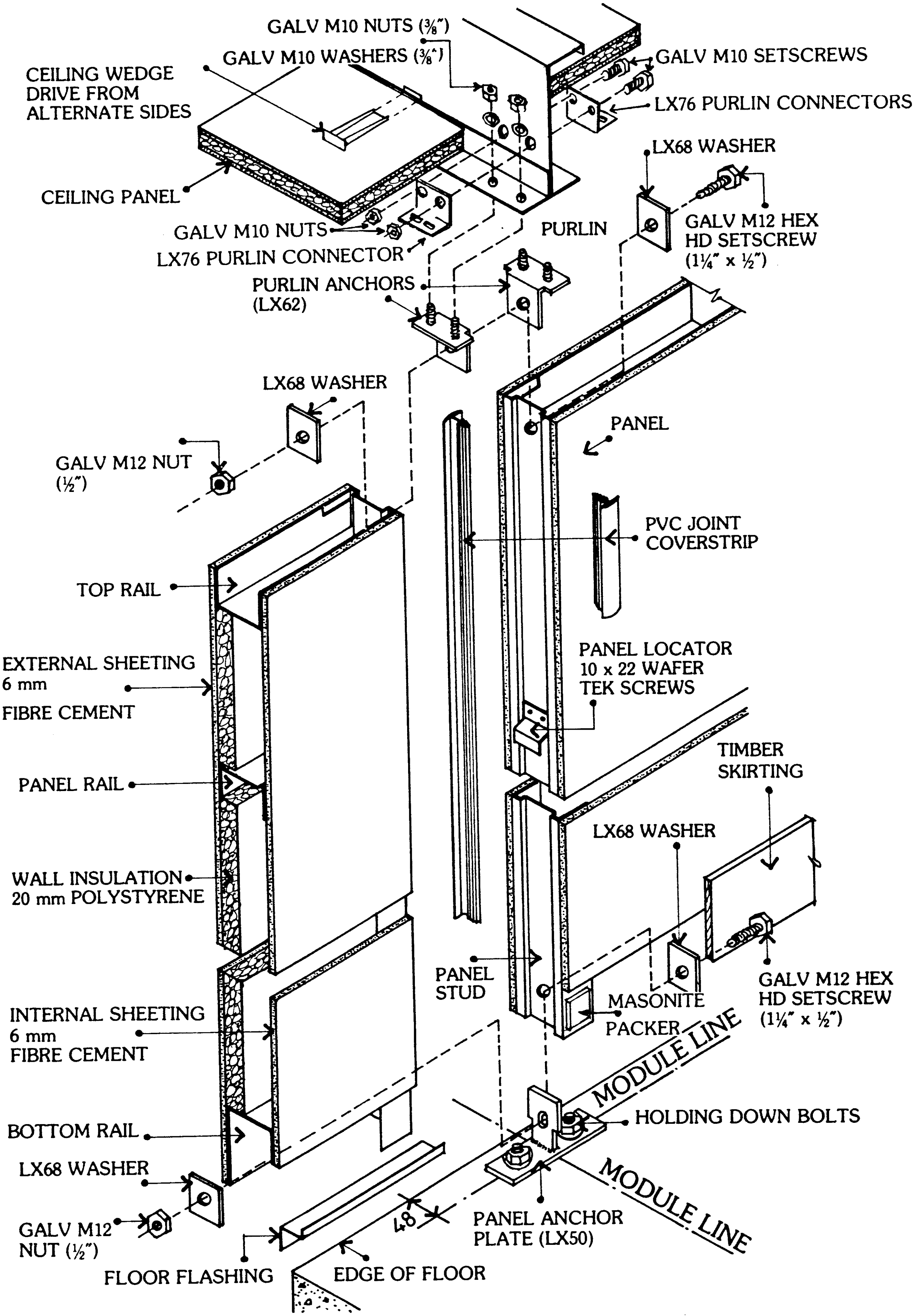


Figure 1. The Logan Unit System
(Courtesy Logan Units P/L)

Most Logan Unit houses have low pitched roofs and use graded purlin roof construction. This is particularly suited to panelized building techniques, as the purlins can be located directly over the studs and bolted to them. In this way the studs provide direct support for the purlins, for both gravity and uplift forces, and the purlins provide lateral support for the transverse walls.

The purlins are Z-shaped but with a double bottom flange, as shown in Figure 1. The insulated ceiling panels sit on top of the bottom flange and are wedged in position. There are a number of holes along the length of each purlin to allow services to pass through.

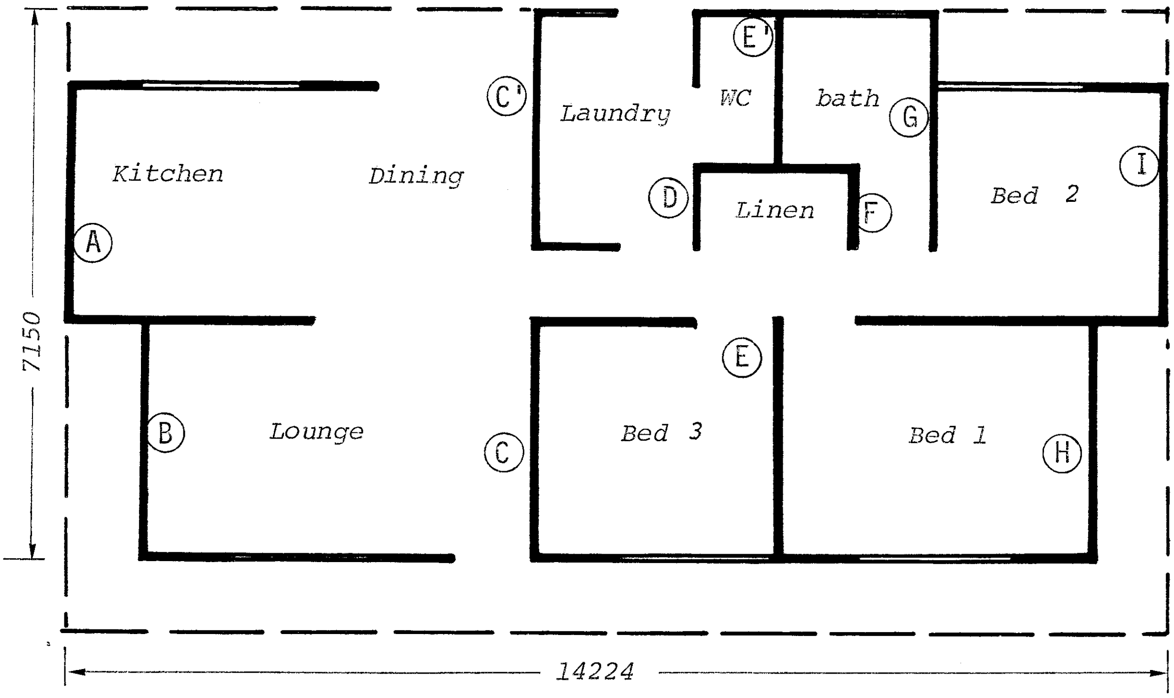
2.2 The Test House

The house tested was the "Carnarvon" design, having plan dimensions of approximately 14 m x 7 m. It was actually 14 modules long and seven wide at its greatest dimensions, Figure 2 shows the floor plan and elevation.

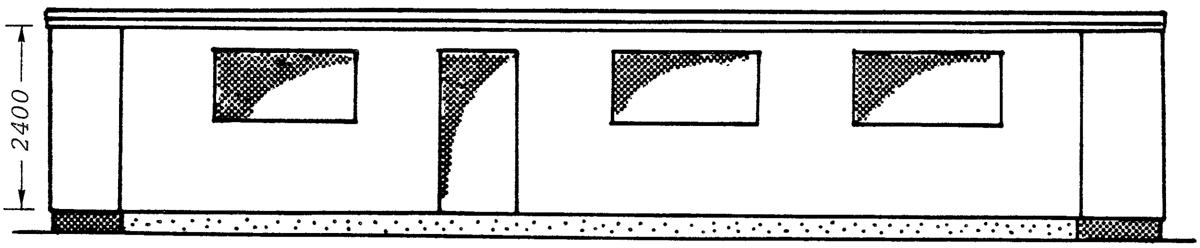
The house and its components were designed to withstand cyclone winds in category 1 terrain. This exposure can be described as probably facing the sea, but certainly having at least one direction which was not protected by surrounding buildings, terrain or vegetation. Details of the design loads associated with these conditions are given in Section 4.

Construction of the test house was as previously described, and as shown in Figure 1. The studs and rails were formed from 1.15 mm thick Zincalume steel. Graded purlin roof construction was used. As the purlins were located over the studs, at module spacing, seven were needed. They varied in height from 150 mm over the outside walls to 300 mm at the ridge, increasing by 50 mm per purlin. The inverted T shape of the bottom flange was spot welded to the 1.55 mm thick upper portion at approximately 170 mm spacing.

The design for severe cyclone wind conditions necessitated a deeper beam and additional column in the open kitchen area. The 152 x 76 x 4.9 mm RHS beam extended from the stud adjacent to the kitchen window to the wall separating kitchen and living rooms. It was supported at the end of that wall by a 76 x 76 x 4.9 mm RHS column anchored to the slab and bolted to the wall. The beam served the purpose of reducing the span of the purlins from six modules to three modules.



Floor Plan



Elevation

Figure 2. Test house - 'Carnarvon' design

In addition to the above, the manufacturer has two standard additions used for all houses in cyclone prone areas. Purlins are braced laterally and thicker outriggers are used. The purlin bracing consists of bridging pieces extending from the bottom of one purlin to the top of the adjacent purlin. These light gauge steel channel sections were spaced approximately 1800 mm apart in the test house and were secured in position by tabs through slots in the web of each purlin.

The normal method of supporting the eaves overhang (one module in width) is by means of 102 x 51 x 2.3 mm RHS outriggers. Each outrigger passes through a square hole in the purlin over the external wall and is bolted to the web of the next internal purlin. They are located at single module spacing. For cyclone areas, a heavier gauge purlin is used,2.8 mm thick colour coded green.

The roof sheeting on the test house was Zincalume steel Kwikclad. It has a profile similar to Lysaght's Trimdek. The sheeting, which was continuous from eave to eave was attached to the purlins with 14 x 50 mm power driven screws, one screw per rib. The screws were fitted with the appropriate saddle shaped steel cyclone washers as well as the necessary neoprene washers.

It was considered unnecessary to include in the test house any details that would not contribute to its structural strength. Thus no doors or windows were fitted, no plumbing or electrical wiring was installed and no benches or cupboards fitted.

The ceiling panels were approximately 30 mm thick sandwich construction consisting of 4.5 mm fibre cement sheets separated by a polystyrene foam core. To allow them to be fitted easily, they were about 10 mm narrower than the distance between webs of the purlins, and were wedged in position against the bottom flange of the purlins.

3. LOADING AND INSTRUMENTATION

High speed air moving past a house generates uplift loads on the roof and drag and suction loads on the walls. These also combine to place an overturning moment on the house. The aerodynamics of the wind/house interaction are well documented by Holmes (1980). In this instance, the effect of the wind was simulated by directly applying lateral and uplift loads to the house. The response of the house was determined by reading deflections as the loads were applied.

3.1 Loading System

The loading system was designed to simultaneously place uplift and lateral loads on the house. The lateral loads acting on the building consisted of pressure loads normal to the windward walls and suction normal to the leeward walls. There were also lateral components of the internal pressure which had no net effect on the house as a whole, and the suction forces on the side walls also cancelled. The net lateral load on the house was therefore distributed over the long sides of house. The net uplift on the roof structure consisted of aerodynamically induced suction on the top surfaces of the roof and internal pressure acting upwards on the ceiling.

For the purpose of testing, lateral forces were placed on the house at the top of the wall. These forces were equivalent to one half of the total lateral wind load, assuming that half of the lateral forces are carried to the top of the walls and the other half to the bottom of the wall. Wind tunnel pressure distribution graphs show that the errors introduced by this assumption amount to less than 5%. The full lateral load was applied to only one side of the house, whereas often in reality much of the suction would have been placed on the leeward side and most of the pressure on the windward side. However, it is possible to envisage the full lateral load being placed on the windward wall with some combinations of window or door openings giving rise to appropriate internal pressures. In previous houses tested, the trussed roof enabled both the windward and leeward walls to act as a unit, but with the Logan Unit house there was no such connecting unit. This had some bearing on the lateral load distribution within the house as discussed in Section 5.2.

The lateral loads were applied using the apparatus shown in Figure 3. The forces were generated by a hydraulic ram (a) which applied tension to a cable. This cable passed over a pulley (b) and through the house to a load spreader (c). Three such loading frames, each equipped with spreaders distributed the lateral loads to twelve points at approximately one metre spacings over the width of the house. As the top of the wall had as its main structural element a light gauge steel channel section, a 20 x 20 mm steel RHS was used to even out bearing pressures at the twelve load points.

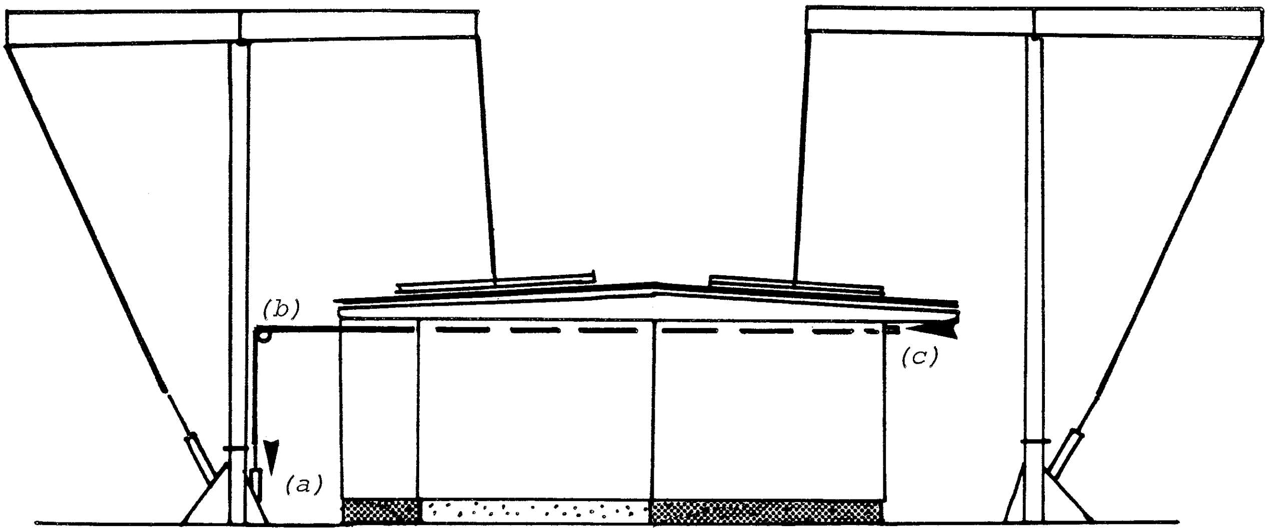


Figure 3. Lateral loading apparatus.

The same loading frames were used to apply point loads immediately adjacent to transverse walls, to find the bracing behaviour of those walls. These point loads were applied to the tops of all potential bracing walls at stages throughout the construction of the building to determine the bracing stiffness of the walls as well as roof and ceiling panels. Some modifications were necessary to apply the failure loads to individual walls, and these are described in Section 6.4.2.

The uplift forces in the test could only be applied to a 4.2 m long section of the house, due to equipment constraints. The test section was chosen to maximise the bending moment in the purlins, and as a result was positioned over the 5 m purlin span in the lounge room. The total uplift force, consisting of the sum of internal pressures and external suction was applied to the underside of the roof sheeting. In a high wind event, this would be possible in the event of perforation of the ceiling lining or loosening of a ceiling panel. The loads were applied using a complex load spreader and the tee shaped loading frame shown in Figure 4.

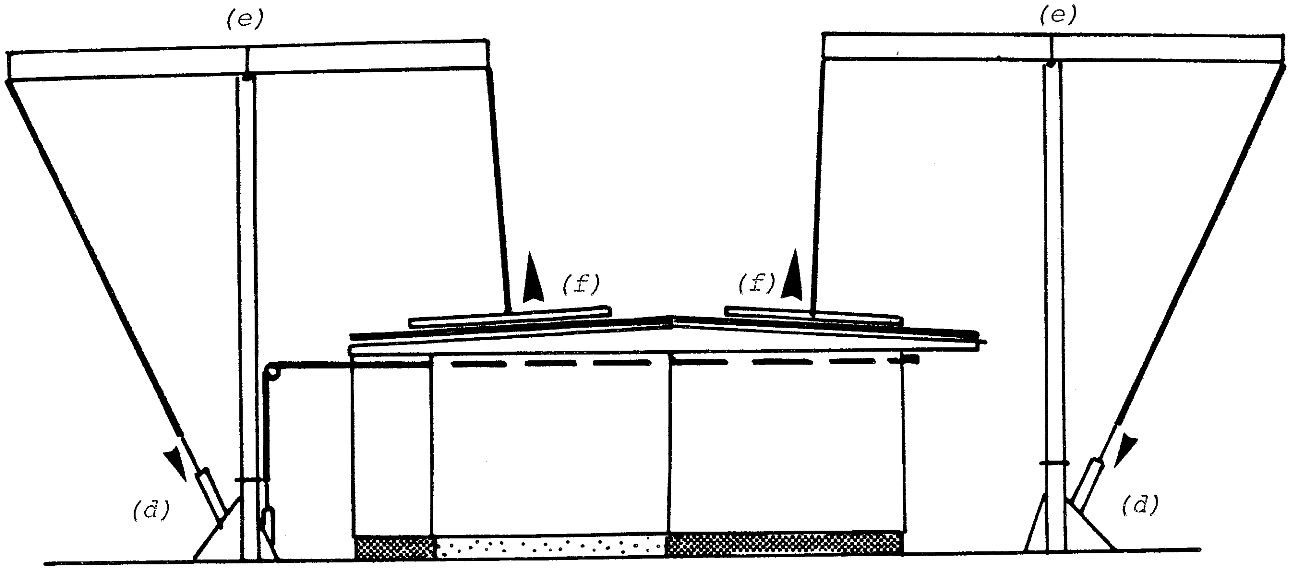


Figure 4. Vertical loading apparatus.

In this case the hydraulic rams (d) pulled downwards on one end of large "see-saw" beams (e). The other end of these beams lifted the load spreaders (f) which distributed the uplift loads evenly to the roofing over a total area of 32 m². This area was the full width of the house and covered a 4.2 m section of the length of the house. The load spreaders consisted of a large number of inter connected steel beams, loaded in the centre and carrying load at each end. A total of 6 hydraulic rams were used and the load spreaders had 54 load rods passing through the roof sheeting to nine timber beams pulling upwards on the underside of the roof sheeting. These timber loading devices were immediately adjacent to the purlins, and ensured that the purlins were loaded through the roofing screws. The weight of the steel load spreaders was deducted from the measured loads to give true uplift on the roof.

3.2 Load and Deflection Measurement

In order to interpret the behaviour of the house and draw conclusions on the load transfer mechanisms within the house, the applied loads and resulting house deflections had to be accurately measured and recorded.

Loads were measured using strain gauge type load cells that were placed in line with the applied cable tensions at the load spreaders. This enabled the true applied lateral force on the house to be found and the true uplift to be calculated. The load cells could be connected either to an analogue indicator or to a digital computer. For tests that involved the manually controlled application of load, the analogue indicator was used, and during sequences of cyclic loading the digital computer received the force measurement information.

Deflections of the house were measured at over 40 locations on the structure, with the datum provided by independent scaffolding on the front and back of the house. The deflection measuring transducers were fixed to this datum with magnetic bases, and their outputs were relayed to a micro computer for processing and storage. The system is depicted in Figure 5, and has been described in detail by Boughton (1983).

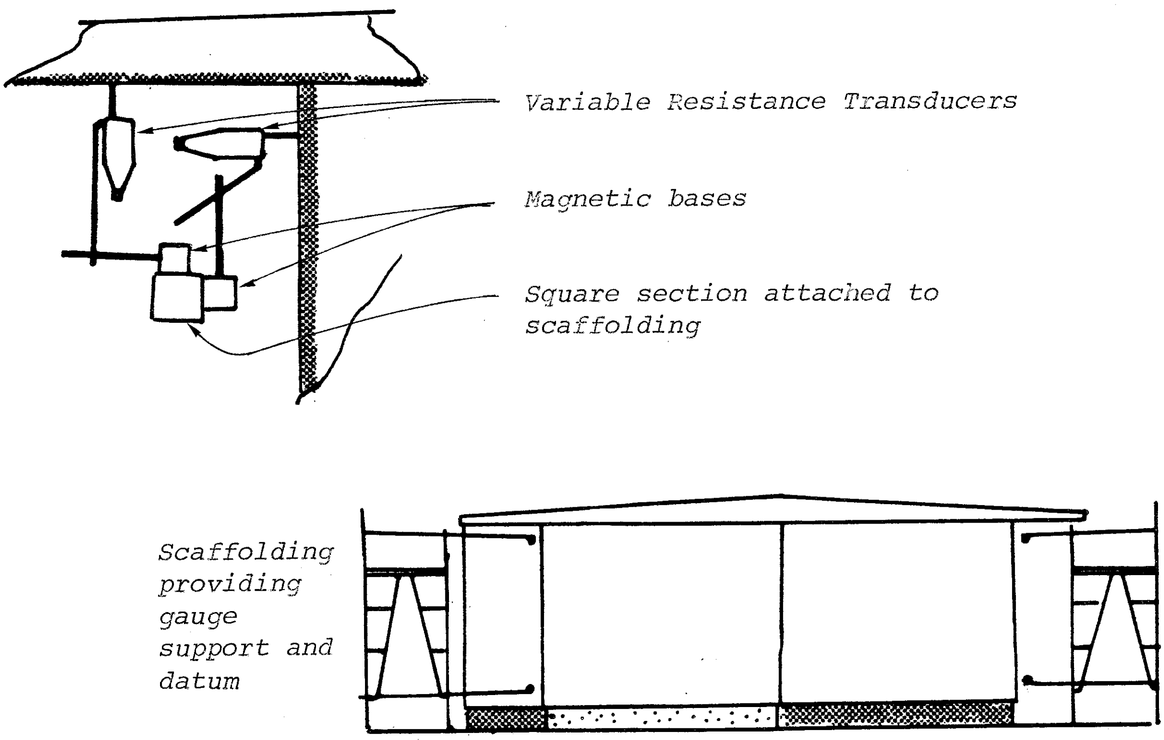


Figure 5. System for measuring deflections.

Most of the deflection transducers were positioned at approximately 1 m intervals along the top of the northern and southern walls. There were only 4 transducers at the base of the walls, and these were located near heavily loaded transverse walls. Other transducers were fitted so that they monitored movement of walls near the centre of the house. Lengths of light steel box sections were bolted to transverse walls near the centre of the house and transmitted deflections through the external walls to enable them to be measured relative to the same datum as the rest of the measured deflections on the house. This is illustrated in Figure 6. Due to the modular construction of the walls there was often a marked difference in response between the two ends of the transverse walls. The remote measuring technique described above was able to confirm and quantify that effect.

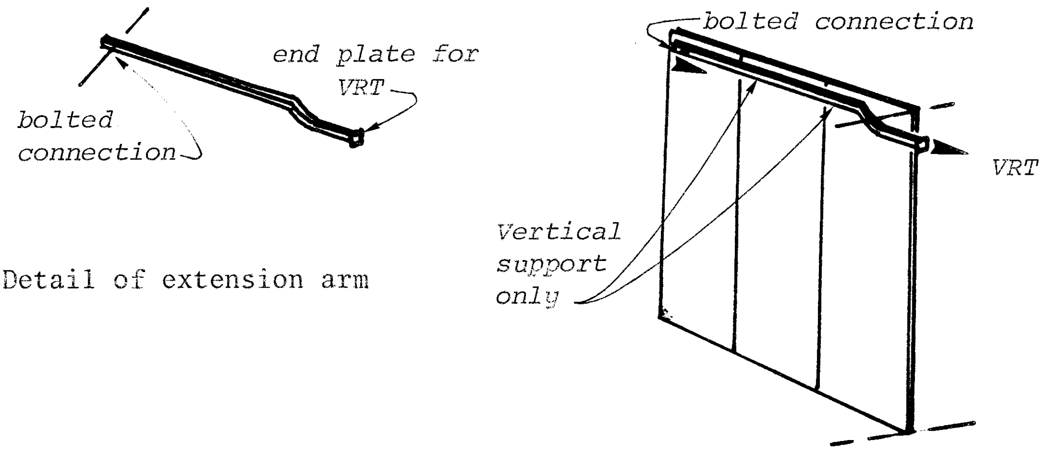


Figure 6. Remote measuring technique.

Other transducers were used to measure the uplift of the roof at the end of the eaves and adjacent to the external wall. These proved most informative during the performance of the cyclic testing, as they enabled failure patterns to be identified.

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.

3.3 Cyclic Loading

As the house was designed for construction in cyclone-prone areas, series of cyclic load tests were performed, simulating the prolonged aerodynamic buffetting that a house receives in a tropical cyclone. These tests were instituted to assess the fatigue susceptibility or tendency to degradation of structural elements under repeated loading, and were based on guidelines established in the Experimental Building Station's Technical Record 440 (1978).

The cyclic tests consisted of a series of loadings, with a prescribed number of cycles from zero to 5/8 design load and back to zero, another set from zero to 3/4 design load and back to zero, and finally a set of cycles from zero to design load and back to zero. In practice, to ensure satisfactory operation a small residual load had to remain at the end of each cycle. This was always less than 10% of the maximum applied load for that cycle.

The uplift forces were applied directly to the roof sheeting, and were therefore the loadings for areas undergoing highly fluctuating aerodynamic loads, as set out in Technical Record 440:

Uplift loadings	8000 cycles	0	to 5/8 design load to	0
	2000 cycles	0	to 3/4 design load to	0
	200 cycles	0	to design load to	0

The lateral loads however fluctuated less rapidly as they reflected the turbulence in the air stream rather than the more severe structure induced turbulence. Thus a smaller number of cycles was set out in Technical Record 440:

Lateral loadings	800 cycles	0	to 5/8 design load to	0
	200 cycles	0	to 3/4 design load to	0
	20 cycles	0	to design load to	0

In order to accurately simulate the total effect of a cyclone on the house, the uplift and lateral loadings were implemented simultaneously. Throughout the sequence ten uplift cycles were required for each lateral load cycle. At a signal from the computer a hydraulic pump was started, and the loading of both uplift and lateral force rams was commenced. Actually applied lateral and uplift loads were separately monitored by load cells and each set of rams was isolated when the appropriate maximum load was reached. The deflections of the house were then read and the rams were unloaded. More readings were taken at the minimum load. Then the lateral load rams were isolated and nine uplift cycles were performed. For the tenth cycle the lateral load rams were again enabled thus giving a sequence of 1 combined load cycle, 9 uplift only cycles, 1 combined load cycle, ... until the prescribed number of cycles had been completed. As most tests involved large numbers of cycles, deflections were read at selected times only during the test. Typically 20 to 30 sets of deflection readings were taken for each set of load cycles.

4. WIND LOADS ON THE HOUSE

The structural components of the Logan Unit House consisted of a large number of panel elements and fixings, each of which was designed according to an Australian Standard appropriate at the time. As the evolution of the system took some years, up to three different editions of the Wind Forces Loading Code could have been used for the design. In order to standardise on the loadings adopted as design loads for the purpose of the tests, the loads calculated using AS 1170 Part 2 (1981) were chosen as design loads.

4.1 Code Determination of Loads

The house tested was irregular in floor plan, which implied that the wind loading it attracted would have been a function of wind direction and orientation of the house. As the house had been designed for terrain category 1 condtions, it was assumed that it would usually be oriented with the larger windows in the lounge and bedrooms facing the views that accompany a terrain category 1 approach. In these conditions the wind approaching the house from the opposite direction may travel over terrain category 3, 2½ or at worst terrain category 2 country. Thus it was assumed for the purpose of testing that the wind approached the house from the front. The lateral loads were maximized with the wind normal to the length of the house. Having decided upon the loading direction it remained to calculate the loads on the house.

During the course of testing, the Standards Association of Australia issued a new edition of the Wind Loading Code, AS 1170 Part 2 - 1983, which represents the current state of the art in wind engineering. Because these provisions were unavailable for the design of the Logan Unit, it was decided to keep using the 1981 edition of the Code as the basis for calculating design loads for cyclic load tests. However, as a comparison design pressures were calculated according to the provisions of both codes and are listed in Table 1. Valid arguments can be made for the use of either code when assessing the test results to failure. The Logan Unit house tested was built to terrain category 1 specifications so design loads were calculated from the following basis.

Regional cyclone wind speed	55 ms^{-1}
Eaves height wind speed	63 ms^{-1}

In calculation of the lateral loads on the house, two cross sectional areas could have been used due to the shape of the floor plan. These were based on
(i) true frontal width of 12.3 m
(ii) greatest projected width of 14.2 m.

The estimated shape of the flow lines around the house indicated that there may be little load on the two projections on the side of the house. A wind tunnel model study tended to support this hypothesis, so the cross sectional area based on the true frontal width of 12.3 m was used in the calculation of lateral load and overturning moment.

For roofs of very low slope, the Wind Loading Code specifies different pressure coefficients across the width of the roof (Table B2.1 of the Code). The highest pressure is near the leading edge, and it decreases progressively away from that edge. This results in a lower overall uplift pressure than would be calculated for a roof pitch in the range of 10^{o} to 15^{o}.

As the test house had a roof pitch of only 3^{o} the design pressures appropriate to that very low slope were used. The resultant uplift load was therefore located in front of the centre of the house, increasing the overturning moment. However as the loading apparatus can only apply a force uniformly, an equivalent uniform load was applied to the roof surface.

An internal pressure coefficient of 0.8 was adopted to simulate the worst combination of loads for all tie down details above floor level.

Average pressures calculated from the codes are shown in Table 1.

4.2 Wind Tunnel Determination of Total Loads

In order to ascertain the effect of the geometry of the building on the loads, a wind tunnel study was performed on a 1/100 scale model of the Logan Unit House. The study utilized a force balance under the model to determine the forces directly. The device and the technique have been described by Roy

(1982), and the study yielded information on the total lateral force on the house, the total uplift on the house and the total overturning moment. These could be converted to an average pressure and are shown in Table 1, compared with the design loads and the actually applied equivalent pressure. A wide range of angle of wind direction was used, but little variation in all three parameters resulted over the range of angles from 0° to 30°.

Table 1

Equivalent pressures on the Logan Unit House
Wind speed 63 ms^{-1}

Calculation basis	Lateral pressure	Uplift pressure
AS 1170 - 2 (1981)	3.11 kPa	3.04 kPa
AS 1170 - 2 (1983)	2.17 kPa	2.86 kPa
Wind Tunnel 0°	1.64 kPa	2.49 kPa
Actually Applied in Cyclic Tests	3.04 kPa	2.88 kPa

4.3 Comments on 'Design loads'

From an examination of Table 1, a significant difference between the design loads calculated from the three different sources can be seen. The large difference in lateral pressures between the two editions of the code (30%) is due to a reduction in both windward and leeward wall external pressure coefficients in the 1983 edition. The reduction in roof external pressure coefficients was less significant and the same internal pressure coefficient in both editions limited the difference in uplift pressures to 6%. However, the wind tunnel determination showed markedly less than either edition. This difference has been confirmed by a series of comparisons of four different models with the 1983 edition of the code and been found consistent for lateral loads and in uplift for similar roof slopes, Roy and Walker (1984).

The loads calculated using the Wind Codes as a basis were derived primarily from measurements or pressures on small areas of the building, whereas the loads obtained from the wind tunnel model were measured total forces for the complete house. It therefore seems valid to attach more importance to code derived loads in assessing the performance of elements and details that receive their load from a few square metres of surface area. These details include purlin fixings, tie down of individual mullions, cladding fastening, and connection of bracing walls to external walls. Similarly it seems valid to attach more importance to the wind tunnel derived pressures when considering overall loads on the house and structural elements required to resist those forces. These elements include the concrete slab, ceiling and roof diaphragms that transmit total lateral loads throughout the building, and possibly bracing walls that as a group carry total lateral loads to ground.

It seemed from preliminary calculations, that the most likely failures during the cyclic loading sequence would be fatigue failures of light gauge steel sections at connections. These elements were carrying loads from relatively small areas of the roof and the walls, so the code determined loads seemed most appropriate. A large scale failure of the entire house appeared unlikely. Thus the house was subjected to the cyclic loading sequence as described in Section 3.3 with the design load calculated using the data in AS 1170/2 (1981) as a basis. In fact the actual loads applied during the cyclic loading sequence fell marginally short of the target figure. The effective lateral pressure applied at design load was 3.04 kPa, 2% less than the required design load. The uplift effective average pressure was 2.88 kPa, 5% less than the design value.

4.4 Other Load Combinations Not Tested

Understandably only one destructive test can be performed on any house. For this house the wind direction that produced the largest lateral load and overturning moment gave something less than the maximum uplift load on the tested portion of roof. This was due to the fact that both the 1981 and 1983 editions of the Wind Forces Loading Code allow progressive reductions in external pressure coefficient from the loading edge of the roof to the leeward edge for flat roofed buildings. Some obvious load combinations not included in the test series were:

(i) If the house had been built with a gabled roof of 10° to 15° pitch, the whole upper surface of the roof would have had a design external pressure coefficient of 0.9 using the 1981 edition of the Code. This would have increased the average uplift for the whole house to 3.59 kPa - a rise of 18%. The 1983 edition gives a 4% increase. Further, if that steeper roof was built using trusses, the problem would be exacerbated. The uplift reaction forces at the end of a truss are significantly greater than any of the uplift reaction forces of the graded purlins in the test house. Thus the results of these tests do not apply to trussed roofs of any pitch, and they would be quite unconservative for trusses of $10\text{-}15^\circ$ pitch supporting lightweight roofing.

(ii) If the wind had come from the end of the building and blown along the long axis of the building from the kitchen end to the bedrooms end, the whole of the uplift test section would lie within the highest two external pressure coefficient regions. [Note that this does not include edge and corner effects - these are average panel pressure coefficients as given in Table B2.1 of AS 1170/2 (1981 or 1983)]. The average roof uplift for the test section under these conditions would have been 3.18 kPa - a rise of 5% over the design pressure for the wind direction assumed. With the wind normal to the end of the building as postulated here the total lateral load and the total overturning moment would have been much reduced (each by over 50%).

The wind direction simulated in the tests, normal to the long axis of the house gave the most severe overall loading on the structure, but it is necessary to be aware that higher design uplift loads on some elements could have been possible had a different wind direction been assumed.

5. NON-DESTRUCTIVE TESTING PROGRAMME

This programme entailed a series of tests that were performed on the house during its construction phase. A series of point and uniformly distributed lateral loads were placed on elements of the house such that no part was loaded beyond its design load. By examining load deflection relationships revealed in this test programme, the role played by different house elements in resisting lateral loads could be found.

The tests proceeded as follows:

(i) After the wall panels had been erected and secured and the purlins attached, a lateral load was applied to the top of each transverse (potential bracing) wall in turn.

(ii) The ceiling panels were then installed, and the previously indicated point loads were repeated. A uniformly distributed lateral load was also applied to the top of the windward wall.

(iii) The roof sheeting was then installed, and the point and uniformly distributed lateral loads were repeated.

In performing these tests, over 21,000 data items were recorded. The analysis of the results of the tests described in (i), gave an estimate of the stiffnesses of individual transverse walls in the house. These estimates together with the results of the tests described in (ii) and (iii) enabled the evaluation of the diaphragm action of the ceiling, and the combined roof and ceiling elements respectively.

5.1 Behaviour of Transverse Walls

These walls were loaded in racking during all of the tests described above. They proved to be very stiff bracing elements, with typical top of wall displacements of 1 to 2 mm at design load. This overall stiffness is due to the following:

(i) very stiff fastening of the base of the wall to the concrete slab. The panel anchor plates and holding down bolts proved to be a very rigid support system, did not permit sliding on the foundation and limited the displacement due to overturning of the complete wall panels to approximately 1 mm.

(ii) efficient bonding of the fibre cement sheeting to the steel wall panel frames. There was no evidence of any movement of the sheeting relative to the frame, indicating that the glue bonding the two together did not slip. This gave each panel rigid characteristics.

The behaviour of the multi-panel walls was also characterised by some closing of the gaps between panels. A PVC joint coverstrip had been jammed into the gap between adjacent panels (see Figure 1), and this strip compressed at the top of the wall giving an uneven distribution of load between the panels. The compression of the strips meant that for a 3 panel wall, the lateral displacement at the top of the wall at the end away from the point of application of the load was about half of that at the load point. This is illustrated in Figure 7. Due to the geometry of the PVC coverstrip, it could only tolerate a small amount of deflection. Thus where the overall deflection of the wall was large, the strip appeared much stiffer and the disparity between the deflections of each end of the wall was less pronounced. The behaviour of walls at loads well in excess of design load during the destructive testing programme verified this mechanism.

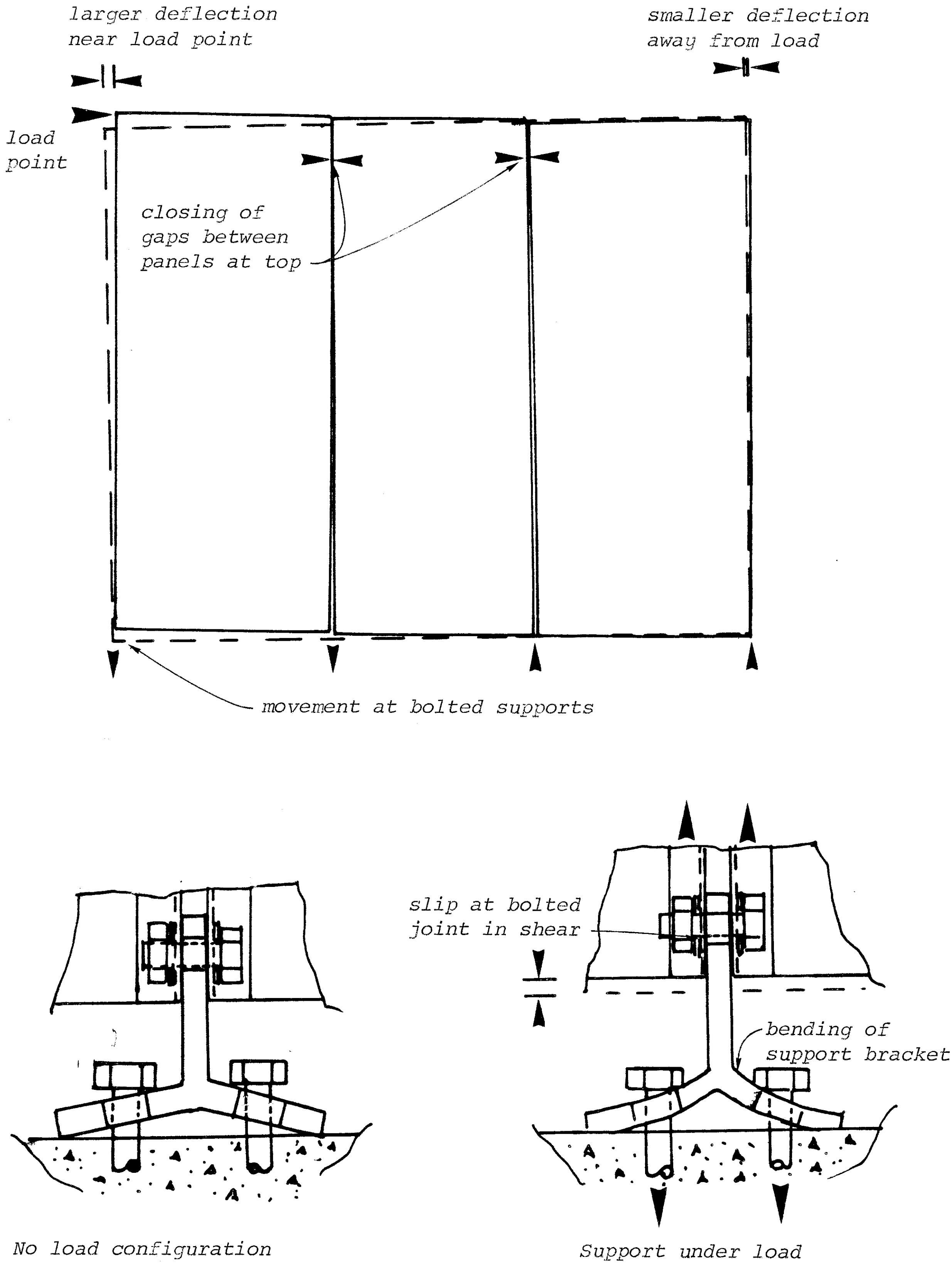


Figure 7. Assumed racking resistance mechanism.

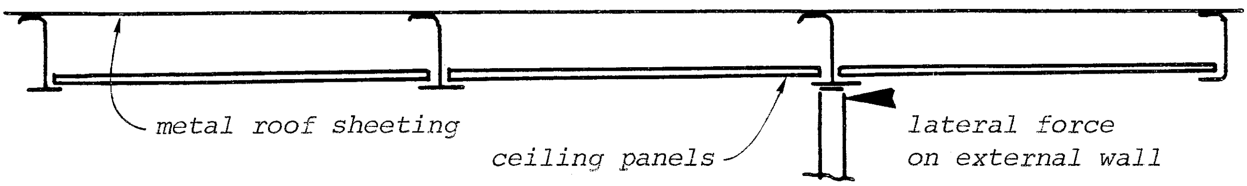
The displacements of the transverse walls of different numbers of panels enabled the racking resistance mechanism to be identified for loads up to design loads. The mechanism identified the principal source of lateral displacement to be overturning of the wall, with most movement at the bolted connections to support brackets at floor level. The bolting between panels proved effective in transferring shear forces between the panels, enabling a multipanel wall to behave as a single composite element. In determining this mechanism, it was assumed that each panel remained rigid, but that the gap between panels could close slightly at the top.

This mechanism was established by assuming different mechanisms and calculating the predicted stiffness of each wall. By comparing actually obtained stiffnesses with predicted stiffnesses, the error in the assumption could be found. It was minimised using the mechanism presented above and illustrated in Figure 7.

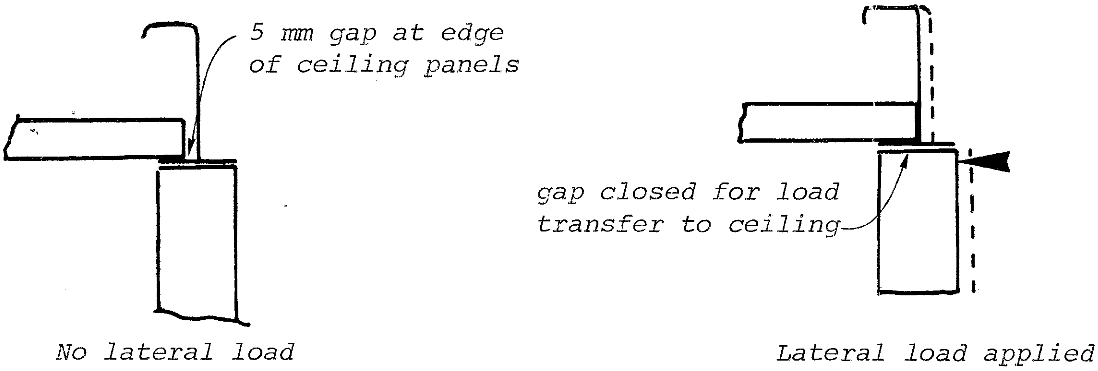
5.2 Diaphragm Action of the Ceiling

By comparing the results of the point load tests on the top of the walls only with those conducted after the ceiling panels had been installed, the amount of load shed to the ceiling diaphragm and carried to ground through other walls could be determined. The actual figures varied from wall to wall, depending on the jointing pattern in the ceiling panels near the loaded wall. Where the ceiling panels were joined immediately above the loaded wall, there was little load transferred to the ceiling at the loads applied. This was the case for 3 walls in the house. On average however, just less than 40% of the load applied to the top of each transverse wall was shared via the ceiling diaphragm with other nearby transverse walls. By comparison, tests on conventional types of ceiling construction have shown in excess of 60% of load carried to other walls by the ceiling diaphragm. This apparently lower effectiveness of the ceiling diaphragm can be attributed to two main effects.

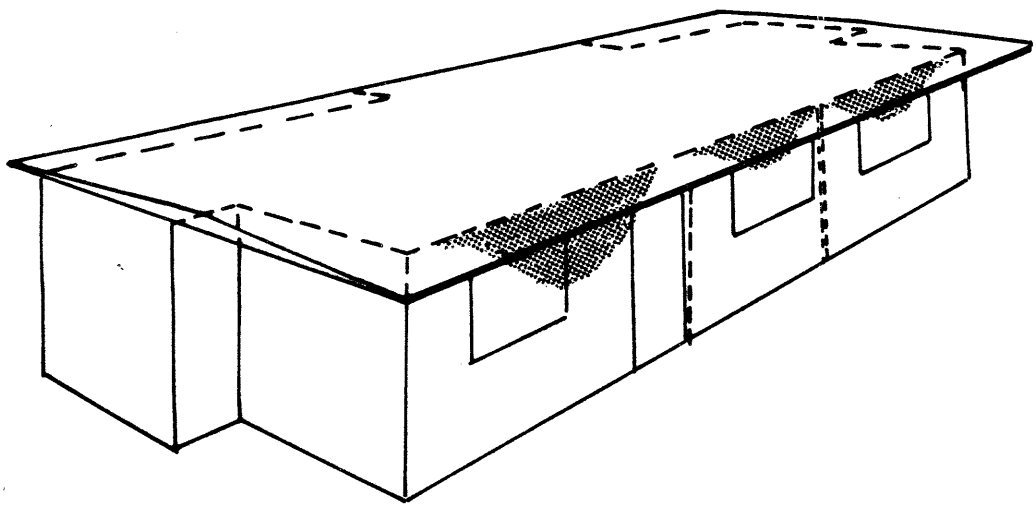
(i) The ceiling panels did not fit snugly between the webs of adjacent purlins and hence the gaps between panels had to close before the diaphragm action could be fully effective. This was made apparent in the uniformly distributed load test, where quite large deflections at the windward edge of the



(a) Roof structure.



(b) Details of load transfer to ceiling panels.



(c) Tributary area for ceiling resistance mechanism.

Figure 8. Diaphragm action of ceiling panels.

ceiling were absorbed within the ceiling diaphragm. It was only over the very open areas that the deflections were transmitted through to the leeward edge of the ceiling. Figure 8 illustrates this mechanism.

(ii) The inherent stiffness of the walls limited the deflections at the top of the wall and hence the relative deflection of the ceiling over the load point. Thus the low percentage of load transferred into the ceiling speaks as much for the stiffness of the walls as it does for the flexibility of the ceiling diaphragm.

The uniformly distributed load test confirmed the mechanism implied above. The very stiff transverse walls attracted load from the windward wall close to them and the flexible ceiling was not able to transmit the loads carried by it very far except in the open living/dining/kitchen area. Over this open part of the floor plan the ceiling was mobilized for the full 6 m depth of the house, but elsewhere its effect was very local. The load sharing was most pronounced where walls were close together and forces were transmitted over relatively short distances.

The combination of the flexible ceiling and very stiff walls effected the required lateral load resistance, but as a result the windward wall bent inwards at the top between transverse walls. Most of this movement was recovered on release of load and so does not present a problem.

The stiffness of the ceiling diaphragm proved variable. As gaps between panels and the purlin webs were closed, the ceiling diaphragm increased in stiffness. This gave the diaphragm the capacity to attract a greater percentage of the load when the applied load was significantly greater than design load. At over twice the design lateral load, the increased deflections had mobilized more of the ceiling and load sharing between transverse walls was more obvious.

5.3 Diaphragm Action of the Roof

The addition of the roof sheeting did little to change the force distribution within the house,which indicates that its function was very much the same as that of the ceiling. In the point loads test, there was little difference in deflections for applied loads at the top of most transverse walls, but where the ceiling panels had been butted over the top of a wall and the ceiling had carried only a small proportion of the total load, the roof attracted the balance. The roof and ceiling diaphragms therefore complemented each other to act as a combined roof structure diaphragm with the roof evening out the effects of the discontinuous ceiling.

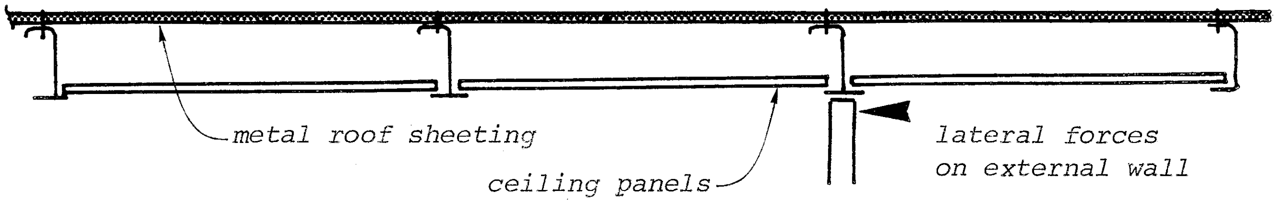
The addition of the roof attracted less than 6% of the total load from direct transfer to bracing walls, but significantly unloaded the ceiling diaphragm. The out of plane bending of the windward wall under uniformly distributed load was halved after the addition of the roof sheeting. This indicated that on average, the ceiling diaphragm and roof sheeting diaphragm carried roughly equal proportions of the lateral load on the windward wall. They each carried this load from the middle of rooms to the bracing walls that could transmit the lateral forces to ground.

In spite of the assistance of the roof sheeting in bracing, the bending deflections of the windward wall were significant. Although the roof sheeting formed a very stiff diaphragm, it was connected to the loaded wall by a relatively flexible detail. The load transfer mechanism between the wall and the diaphragm was via the purlin which could only effect the transfer by bending its web. Calculations showed that approximately 10 mm relative deflection between the roof diaphragm and the wall was required to activate the full load transfer. This displacement was not achieved at any point in the roof. The maximum relative displacement was approximately 5 mm which indicates that the maximum local force transfer to the roof sheeting was about 50%. The average for the whole house was less than 20%.

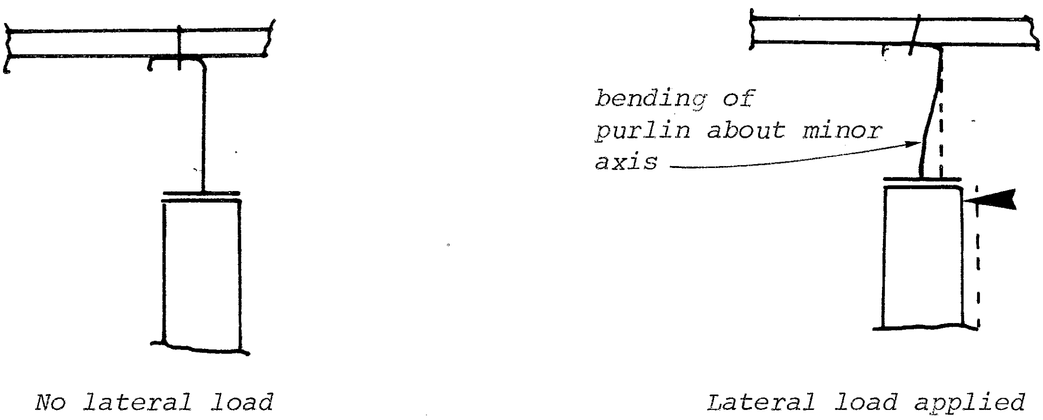
Figure 9 shows the load transfer mechanism postulated for the roof diaphragm. Load transfer into the roof diaphragm was via the bending of the web of one purlin. However load transfer out of the roof-diaphragm was through five purlins. The resultant mechanism was quite flexible in attracting load to the diaphragm, but quite stiff in shedding it. Thus the roof remained positioned above the walls that did not deflect much, and the windward wall moved inwards underneath it.

5.4 Aggregate Lateral Load Resistance Mechanism

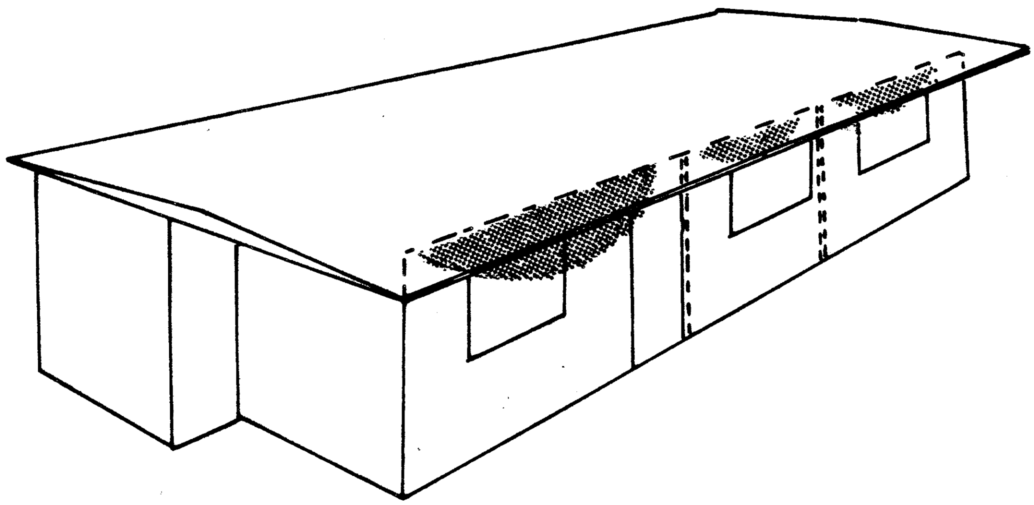
During high winds, lateral loads are applied to the house as a pressure loading mainly directed to the windward wall. Those areas of the wall close to junctions with transverse walls tranfer most of their load directly to the transverse



(a) Roof structure.



(b) Detail of purlin under lateral load.



(c) Tributary area for roof resistance mechanism.

Figure 9. Load transfer mechanism of roof diaphragm.

walls, with the remainder passing to the combined roof and ceiling diaphragm for redistribution to nearby transverse walls. For area away from those junctions the load is carried sideways mainly by the roof and ceiling diaphragms. Both of these diaphragms are characterised by flexible connections to the windward wall which allows some out of plane deflection of the wall and associated load transfer by bending of the windward wall.

The roof and the ceiling diaphragms, while not giving as rigid support as has been found with timber framed construction, functioned adequately to share lateral loads to bracing walls without sign of any permanent damage at loads up to and beyond design load. The very stiff bracing walls limited the maximum total house deflection at design load to less than 10 mm and the average to less than 2 mm. Deflections of these magnitudes would have hardly been noticed by occupants at the time of the loading.

6. DESTRUCTIVE TESTING PROGRAMME

There were three phases in the destructive testing programme, cyclic loading, static loading of the house to failure and static loading of some components to failure.

As previously mentioned, the cyclic loading programme was meant to simulate the buffetting effect of the wind gusts during the period of the cyclone. The number of cycles and intensity of pressure have been explained in Section 3.3. Because of the relatively large volume of oil that had to be displaced for the bank of hydraulic jacks to load and unload the structure, the cycle time was considerably slower than it would have been during a cyclone. The fastest rate of cycling achieved was five per minute. However this slower cycle rate is not considered to have had any effect on the outcome of the tests.

The static loading of the structure to failure was used to determine how much reserve strength it had in case the cyclone contains some wind gusts greater than that considered as design.

In order to maximize the results obtained from the programme some elements not damaged during the test to failure of the house were tested as individual components. Their performance is discussed later in this section.

6.1 Combined Cyclic Lateral and Uplift Loading

6.1.1 Cycling to 5/8 design load

During this loading sequence the roof was subjected to 8000 load and unload cycles and the walls to 800. For nine cycles out of every ten, the roof only was loaded, while the walls sustained only 10% of the maximum applied load. On every tenth cycle the walls and roof were simultaneously loaded to 5/8 of of their respective design loads, and unloaded. Displacement readings were taken after the first wall cycle, the second, fourth, eight, sixteenth, thirtysecond, sixty fourth, ... etc. and 800th cycles.

The walls showed some sign of creep and lack of recovery in the early stages of the test but had quite elastic behaviour throughout most of the twenty eight hour test.

The gauges monitoring the roof deflection showed a change in stiffness of that area directly over the kitchen window. By the end of the test the out-riggers were deflecting twice their initial movement and the purlin over the window had increased its deflection by 30%. This was in contrast with the rest of the house, which showed almost no change in stiffness during the test.

A visual inspection of the house after completion of the loading sequence showed no visible signs of damage, but most of the structural fixings were inaccessable. In view of the later performance of the house it is likely that some tearing of the top rail had occurred at the purlin anchor position over the centre of the kitchen window. The deflection records indicate that the tearing would have started after about 5600 uplift cycles of 5/8 design load and may have been quite advanced at the end of the 8000 cycles.

6.1.2 Cycling to 3/4 design load

Again during this sequence, nine out of ten load cycles were on the roof in uplift only with the tenth load/unload cycle simultaneously on both the roof in uplift and the walls in racking. The roof was subjected to 2000 load and unload cycles and the walls to 200. Displacement readings were taken every 200 roof cycles.

The loaded front wall of the house showed signs of creep throughout the duration of the test, which meant the progressive lack of recovery was roughly equal to the elastic displacement of most gauges. However the lack of recovery was concentrated in the first 150 wall cycles, with the last 50 showing almost total elastic behaviour. The overall lack of recovery varied from 1 to 3 mm. There seemed to be very little movement on the back wall which confirms the behaviour shown in the elastic tests. With no rigid continuous members connecting the two walls as in a trussed roof house, there was room for relative movement between the walls to be absorbed within the panel movement in the ceiling and by bending of purlins. However wall stiffnesses were very similar to those experienced in the cycles to 5/8 design load indicating that little degradation was taking place. Further the wall stiffnesses remained the same throughout the 200 cycles in spite of damage to the roof at the southern end, and stopping and starting the test. It is therefore postulated that little or no damage took place to the panels resisting racking forces. This is in agreement with the lack of damage to walls, encountered in observation of the building.

A very dramatic change occurred during the period between 1200 and 1400 cycles when upward displacement of the roof over the kitchen window suddenly increased by 15 to 20 mm. As the loading sequence was operating unattended at the time, the precise number of cycles completed prior to the damage occurring is unknown. However the pattern of the damage is quite clear. After 1480 roof cycles the loading sequence was terminated and a calibration check of the load cell controlling the uplift cycles was performed to ensure that the roof had not been overloaded. The calibration was within 6% of the previously obtained calibration which was quite satisfactory. The applied roof uplift loads were generally 5% under the appropriate fraction of design load, so the roof could not have been overloaded.

Inspection of the damaged area revealed severe tearing of the top rail over the centre of the window. This was where an angle bracket was used to attach the purlin to the top rail of the wall. Figure 10 shows the detail. The bracket was now broken.

A visual assessment of the force paths showed that the 150 mm purlin over the outside wall was required to resist the entire pressure on the eaves overhang (one module wide) plus a contribution from the roofing spanning to the next purlin. Whilst the normal holding down detail, connecting purlin to a pair of studs, seemed adequate the detail used over the window was unable to resist the applied loads.

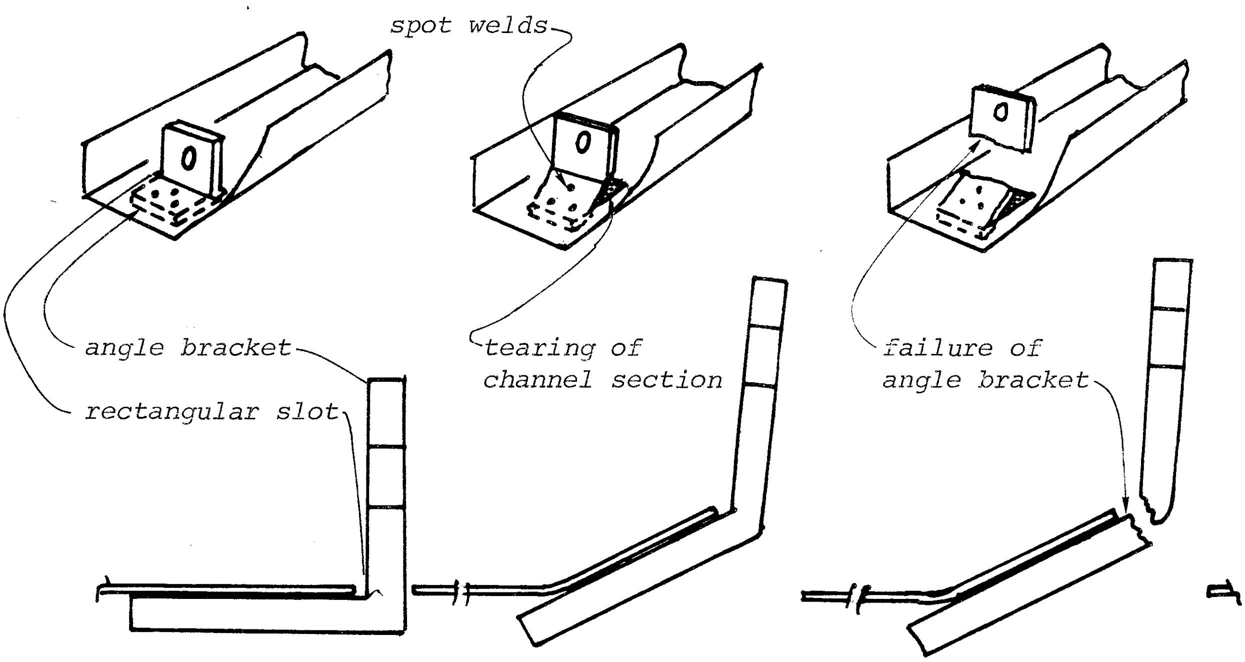


Figure 10. Failure mechanism of angle bracket.

It is postulated that most of the tearing of the top rail occurred during the 8000 cycles of 5/8 design load. The bracket which was spot welded to the top rail resisted the first 1000 cycles of 3/4 design load as a tension member, but the fatigue stresses caused it to break completely somewhere between 1200 and 1400 of cycles in this load sequence. This failure caused the purlin to have to span the complete width of the window (two modules) which overloaded the purlin fixings to the stud at each end of the window but did not cause failure there.

The test was resumed, with a substantial increase in deflection of the purlin over the window. By 2000 cycles the elastic deflection of the roof over the window was four times the value at 1480 cycles. There was a permanent gap of 10 mm between the top of the wall and the purlin. Further inspection showed that the spot welds joining the double bottom flange of the purlin to its web broke at each end of the window.

Although the lounge room window represented the same situation as the kitchen, it did not fail. This was because the horizontal loading gear pierced the wall at the exact location, and was of yoke shape which prevented the purlin from separating from the wall. Obviously, if this type of failure had been anticipated a different loading arrangement would have been used. Unfortunately the extent of assistance given by the loading yoke cannot be quantified, except to say that it would have been substantial.

6.1.3 Cycling to design load

Although failure of an element of the structure had occurred at 3/4 design load, the house was still able to resist design load by redistributing the forces within its hyperstatic framework. This demonstrates the advantage of testing full size houses rather than just components.

Again during this sequence, nine out of ten load cycles were applied to the roof in uplift only with the tenth load/unload cycle simultaneously on both the roof in uplift and the walls in racking. The roof was subjected to 200 load and unload cycles.

This time, the loaded front wall of the house showed little sign of creep with lateral displacements being almost elastic. The back wall showed rather erratic behaviour in the vicinity of the kitchen where the roof support system was extensively damaged.

Deflection of the purlin over the kitchen window increased significantly during this test. Near the completion, the roof was lifting in excess of 40 mm on each load cycle. Residual deflection at the end of the test was about 25 mm. It was obvious that more load was being shed to the studs one module away from the window.

At the conclusion of the test there was about 9 mm permanent gap between wall and purlin over the lounge room window.

6.1.4 Static test to failure

During this test, the loads on both the roof and the walls were gradually incremented at much the same time, until a failure load was reached in either lateral or uplift loading. Each set of rams was a different circuit, and the lateral load circuit was increased first.

The performance of the house in resisting uplift loads was surprisingly linear, but the maximum elastic deflections of the roof were approximately six times those of the walls. The maximum roof deflections were experienced over the kitchen window where the total separation of the roof system from the top of the wall was in excess of 100 mm at the completion of the test. This deflection was accommodated by more failures of spot welds securing the flange of the purlin to the web. This allowed the flange and the web to separate, with vertical reactions still transmitted by the flange in tension. The load was increased until further lifting produced no increase in load. At this point the bottom flange of the purlin had started to tear in two places near a stud on the east side of the kitchen window. To prevent catastrophic failure, the test was terminated. The maximum uplift load sustained was 22.5 kN per load spreader, and was a net figure after allowing for the weight of the load spreader. This gave an average uplift pressure over the loaded area of 3.94 kPa, representing 1.30 times design load as calculated using AS 1170/2-1981.

The racking stiffness of the house was still quite high during this test. The maximum lateral deflection of the top of the loaded wall was 6.3 mm, when the test was terminated at a pressure equivalent 5.0 kPa on the front wall. This is equivalent to 1.61 times design lateral load calculated from AS 1170/2-1981.

Inspection of the house did not reveal any evidence of racking damage, and it was concluded that lateral failure was not at all imminent.

6.2 Modifications to Test House

As the Engineer from Logan Units was present during the test programme, the opportunity was taken to redesign the purlin hold down detail that had failed. The redesign was a relatively simple exercise as the test had highlighted the weakness in the original detail as being the lack of flexural strength of the web of the top rail. The new detail was therefore designed to transfer the purlin forces into the rail by shear rather than by flexure. Figure 11 shows the redesigned detail.

Although the redesign of the bracket was relatively simple, repair of the house was not. However the damaged purlin was eventually removed, the protype holding down bracket was installed and a new purlin was fitted. The house was made complete by replacing the roof sheeting.

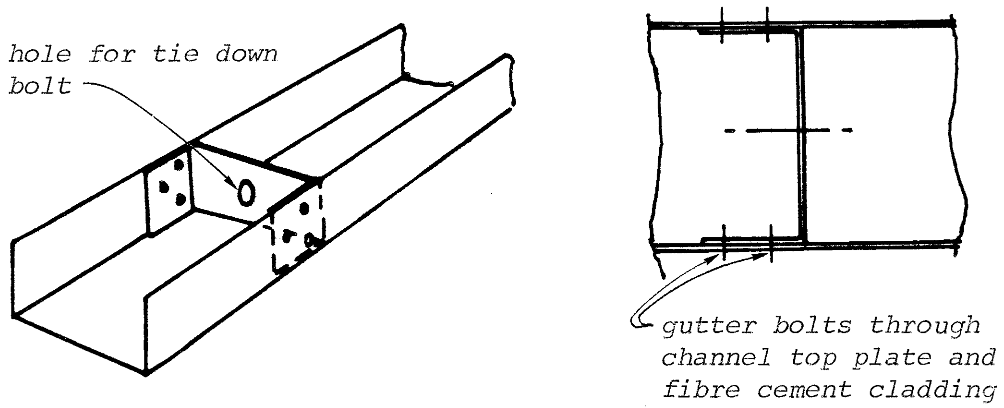


Figure 11. Redesigned bracket for window head.

The new holding down bracket should be regarded as a prototype which was made for installation into the existing house. It was held in position by bolts which of necessity had to be fastened through the cladding. The bracket fastening system will therefore need to be altered to be efficient for factory production. However the altered detail must still have the same action in transferring forces as was used in the prototype.

6.3 Testing of Modified House

6.3.1 Cyclic loading

The addition of the new bracket and purlin posed somewhat of a problem regarding the test programme. Whilst the rest of the house had been cycled 1000 or 10000 times, these new elements were still unstressed.

In consultation with Logan Units' Engineer, it was decided to recycle the kitchen/dining areas of the house using uplift forces only, see Figure 12. This would subject the new elements to the same number and degree of load cycles as the purlins in the lounge area of the house. It was acknowledged that the other purlins in the kitchen/dining area would therefore receive a total of twice the number of cycles, but a close inspection had revealed no evidence of fatigue after the first set. This was to be expected as the span of those purlins was only about three metres compared with five metres for similar purlins in the lounge area. The roof sheeting in the kitchen/dining area would also be subjected to a total of 20000 load cycles, but this was considered to be of little consequence because of the method of loading the sheeting. The uplift forces were applied immediately adjacent to each purlin, thus although the roofing had to transfer all that force there were virtually no bending stresses on the sheet. Laboratory tests on other roof sheeting have shown that bending moment plays a very significant part in the failure of roof sheeting.

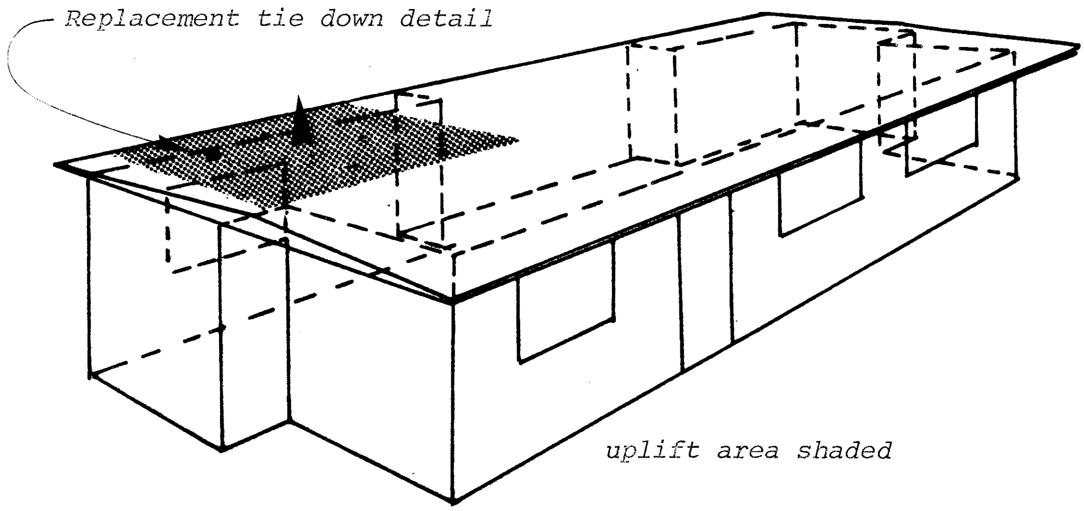


Figure 12. Area of roof loaded to stress new tie down bracket.

8000 load/unload cycles of 5/8 design uplift load was therefore applied to the 4.2 m length of roof over the kitchen/dining area. No other uplift loads or lateral racking loads were applied.

The roof behaved almost elastically throughout these tests, with the new bracket showing only 0.6 mm creep at the end of the cycles. Other deflection measurement points showed excellent recovery.

The test programme was continued with 2000 load/unload cycles of 3/4 design uplift load being applied to the kitchen/dining area only. In contrast with the previous sequence of 3/4 design load, there was little or no damage to the house during this test. Also there was little increase in elastic deflection compared with that measured during the 8000 cycles.

200 load/unload cycles of design load were then applied to the 4.2 m length of kitchen/dining area roof. Load deflection curves all indicated little change in stiffness and good recovery after each cycle. The one exception was an outrigger which showed a sudden jump in deflection from 6 to 8 mm. As there were no signs of damage the deflection was assumed to be movement of a bolt within a hole.

At the completion of this sequence all elements of the house within the test area were considered to have had at least 10200 cycles of uplift load and 1020 cycles of lateral load. The redesigned holding down bracket was still securing the new purlin in position, and the purlin was flush with the top of the wall.

6.3.2 Static test to failure

This test involved uplift over the full width of the building at the living end of the house plus lateral racking forces, that is, the same loading arrangement as described in Section 6.1. The loads were applied incrementally with the lateral load increment being applied first and then the uplift load. Deflection measurements were recorded at each increment.

The response of the walls up to the failure load was very similar to that experienced in the previous combined loading destructive test. At loads between half design load and design load, the deflection transducers on the back wall started to register movement. At this stage transverse walls near the back of the house started to carry racking loads and hence the overall stiffness of the house appeared to increase. At loads greater than the design load, most horizontal deflections remained linear. At the failure load this was not the case for transducers measuring horizontal deflections of the top of the wall over the lounge room window. Prior to failure, the transducers in that area extended out of their range, and it was considered to be too dangerous to reset them. The actual deflections of those points near the failure load were not recorded. The deflections over the lounge room window were significantly higher than those over the main bedroom window, whereas in previous tests they had been much the same. This suggests that the connection between the top of the wall and the roof structure over the lounge room window had deteriorated and was not adequately restraining the top of the wall. This was supported by visual observations which showed that the purlin had separated from the top plate by 20 mm prior to failure.

The uplift behaviour of the house was quite different from its behaviour in the previous test to failure. The south side of the house with the new purlin and attachment detail showed linear elastic behaviour up to 3.52 kPa on the loaded area (1.16 times the designated design pressure). However at this pressure a well defined yielding was recorded at all measurement points on that side of the house. It was the result of failure of spot welds between the flange and web of the purlin. Upwards displacement per load increment was five times the amount recorded before the yielding. Near to the failure load, the purlin had lifted about 10 mm above the top of the wall, but the new connection detail over the kitchen window was still intact.

The north side of the house yielded at 3.96 kPa (1.30 times designated design load), with the main deflection increase over the lounge window. However at the next increment of load (to 4.4. kPa) the roof rose 14 mm over the centre of the window and 12 mm over the studs at each end. Although the actual detail could not be viewed at the time, because of the presence of lateral loading equipment, it was presumed that the bracket over the lounge room window failed in a similar manner to the one over the kitchen window. After completion of the tests the bracket was examined and found to be broken. Some of the spot welds on the purlin over the window also failed at this load increment.

The static loading increments of uplift and racking were continued until the purlin over the lounge room window was unable to resist the increase in load, and the bottom flange tore away from the web. It was accentuated by the lateral force which had moved the window head 80 mm sideways. The maximum applied pressure on the front walls was 6.92 kPa and on the roof was 6.06 kPa, representing 2.2 and 2.0 times the respective designated design pressures.

At this stage the roof had lifted over the lounge considerably with a gap of approximately 60 mm between the wall top and the purlin over the window, the two purlins spanning the lounge room buckled at points approximately 400 mm off the centre of the loaded area, where there were joints in two out of the 3 ceiling panels in the lounge room. The unjointed panel was bending visibly in the vicinity of the buckling failures. The head over the loung room window had moved upwards approximately 50 mm and the purlin over the window was severely twisted and torn. The RHS beam picking up the mid points of the six metre purlin span across the kitchen/dining area was showing significant hogging, but no floor fixings other than the one under the southern end of the RHS kitchen beam were showing severe signs of stress. The RHS in the kitchen/dining area was held down to the floor on the southern end by a stiffened hold-down detail which was designed to carry half of the load of the RHS as well as the load from the outrigger immediately above it. That fastener was showing signs of extreme deformation and the base of the mullion had lifted approximately 6 mm. However, it was still satisfactorily holding load. The purlin over the kitchen window had bent visibly and lifted approximately 10 mm. The new connector holding the purlin to the window head had not failed, although the fibre cement sheeting on the outside of the head had cracked and the screws through the sheeting into the top plate had caused delamination of the fibre cement sheeting. The top plate channel had lifted a little relative to the external sheeting, and complete separation of the channel from the window head may have been imminent had loading continued.

After the load was released, the bending of the RHS in the kitchen/dining area recovered, but residual damage remained over both lounge and kitchen window heads, at the mullion supporting the kitchen RHS and in the lounge room window head top plate. The house showed little or no residual damage due to racking forces on transverse walls.

6.4 Lateral Load Tests After Failure

6.4.1 Uniformly distributed lateral load

As the house had suffered very little racking damage during the combined lateral and uplift load tests, it was decided to load 2/3 of the length of the front wall with a uniform lateral load. It would have been too dangerous to load the lounge room wall, so the load was applied to the length from the front door west to the bedroom area. No uplift forces were applied.

The test was quite uneventful as the capacity of the hydraulic rams was reached before there were any signs of failure, and the load-deflection curves for displacement along the front wall were still linear. The force applied was the equivalent to 7.52 kPa over the loaded area or 2.42 times the designated design load. It is estimated therefore that the racking strength of the whole house is significantly greater than the forces applied during this test.

The history of the test was similar to the history of other racking tests with load being carried by the back part of the house only after the design load had been reached. The unloaded end of the house also experienced some lateral displacement during the test which indicated that the ceiling and roof diaphragms were capable of transferring load to other transverse walls. The elastic deflections at the rear of the house were very much smaller than those at the front of the house indicating that the bulk of the load was resisted by the three walls (i) between the lounge and bedroom 3,(ii) between bedroom 1 and bedroom 3 , and (iii) the western external wall on bedroom 2. These walls showed slight evidence of lifting at the front but the movement was less than 2 mm. Other than that, there was no visible sign of movement and the maximum transverse wall displacement was 7 mm at the top or less than height of wall/300. The maximum inwards movement of the loaded external wall relative to the corners of a room was 11 mm or less than span of wall/350, so the diaphragm action of roof and ceiling proved effective at these high loads.

6.4.2 Tests on individual walls

Walls A, I and G (see Figure 2) were selected to be tested as individual bracing walls. Each was three modules long, but wall G had a doorway as well. The ceiling adjacent to each wall was cut for the length of the room, to prevent any diaphragm action transferring force to adjacent walls.

At approximately 14 kN the top rail of wall A started to crush. At an ultimate load of 26 kN the top plate buckled adjacent to the point of application of the load. At this stage the wall had moved 5 mm at its free end and about 8 mm at the loaded end. There was no evidence of damage to the detail securing the panels to the ground.

Wall I was loaded similarly to wall A. It also failed at 26 kN, but in this case failure was a local buckling of the stud at the load application point.

This was considered to be an unrealistic failure as it was initiated by the loading system.

At wall G the racking force was applied to the doorway and transmitted to the three panel long wall. However the door frame was incapable of carrying more than 16 kN as the light gauge steel jamb started to crush. A bridging piece was inserted to effectively bypass the doorway. The load was then applied and an ultimate of 30 kN was achieved. The failure of the loaded wall was similar to that experienced by wall A in that the top plate channel started to buckle. In the process of achieving that load, the timber load spreader crushed the two mullions, the timber bridging piece tore the purlin adjacent to the load point, and the door frame completely buckled, allowing the intersecting wall against which the load was applied to move over 50 mm towards wall G. This large movement caused failure of the fibre cement cladding of the intersecting wall. Thus, although the test demonstrated the high strength and stiffness of the wall in racking, it also dramatically illustrated the problems associated with getting such high loads into the wall using light steel sections.

6.4.3 Fibre cement cladding

Fortuitously, the panels for the test house were fabricated during the period when asbestos-containing sheet was being phased out by its manufacturer. Thus there was some uncertainty as to whether all the panels were made with the new asbestos-free board. On completion of the tests a sample piece of board was identified by James Hardie and Coy as containing asbestos fibre. It has therefore been assumed that many of the panels would have been clad with that older type of sheeting.

As the new asbestos-free board does not necessarily have the same strength properties as the previous board, the decision was made to conduct a comparative racking test on a wall of the new construction.

An isolated wall consisting of three panels was constructed in the laboratory. It was meant to simulate a typical wall in the house, and can be compared with walls A or I described in Section 6.4.2.

After some preliminary stiffness tests the wall was loaded in racking until failure, at 22 kN. Whilst the metal frame started to crush in a manner similar to the other tests, it was also associated with some delamination of the cladding. This did not happen in the house tests.

It is not clear from that single test whether the reduction in racking load is actually due to the different board or just to normal variation that occurs in test programmes. The different failure mode indicates the former. However if that test represents the likely performance of the new cladding material, it would have little if any effect on the test results obtained for the house.

7. CONCLUSIONS

The tests on the Logan Unit have once again demonstrated the benefits of testing entire houses rather than individual components. This is amply demonstrated by the performance of the purlin over the kitchen window, which was badly torn at the end of the 8000 cycles of 5/8 design load yet it managed to redistribute loads for the remaining 2200 cycles and then resist 1.3 times design load. It is unlikely that such redistribution would be able to occur during a simple laboratory test.

The following conclusions can be drawn from the tests on the Logan Unit.

(a) The house in its original condition failed at combined racking and uplift loads of 5.0 kPa and 3.95 kPa respectively when the purlin over the kitchen window was unable to resist a further increase in load. This is equivalent to 1.61 times design racking load and 1.3 times design uplift load.

(b) The racking stiffness of the house was very high, as all wall panels acted as bracing elements. The ultimate racking strength of the house was not determined as failure of the window head detail in uplift prevented further lateral loads from being applied in that area.

(c) The ceiling panels and roof sheeting transferred load adequately in diaphragm action, but the ceiling had to take up some slack before becoming fully effective, and the flexible connection between the roof sheeting and the windward wall decreased the overall efficiency of load transfer.

(d) It is likely that the holding down detail over the lounge room window (identical to the original detail over the kitchen window) would have failed earlier if it had not been restrained by part of the lateral loading rig. It is also likely that the purlin over the double door opening in the dining area would have failed had it been fully loaded.

(e) The new purlin and modified holding down bracket over the kitchen window resisted 10200 cycles of load without any signs of damage.

(f) Spot welds on the new purlin started to fail at 3.52 kPa (1.16 x design load) but redistribution of the applied load allowed a total of 6.06 kPa to be applied without serious failure.

(g) The house (with the modified purlin holding down detail) eventually failed at an uplift pressure of 6.06 kPa (2.0 x design uplift load) and a lateral pressure of 6.92 kPa (2.2 x design lateral load) when the purlins in the lounge failed in bending.

(h) The hold down of panels to the concrete slab was adequate in so much as the only sign of distress was at the mullion supporting one end of the RHS beam in the kitchen.

(i) The racking strength of a three module long wall is about 26-30 kN.

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