THE JAMES COOK UNIVERSITY OF NORTH QUEENSLAND
CYCLONE STRUCTURAL TESTING STATION

# CYCLONE TESTING STATION

# TESTING A HIGH SET HOUSE DESIGNED FOR 42 m/s WINDS

## Part 1 — Preliminary Results

TECHNICAL REPORT No. 19

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CYCLONE TESTING STATION

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G.N. Boughton
G.F. Reardon

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# TESTING A HIGH-SET HOUSE

# DESIGNED FOR 42 m/s WINDS

## Part 1 - Preliminary Results

G.N. BOUGHTON*

G.F. REARDON**

SUMMARY

Housing forms a very complex structural system. Wind forces are resisted by intricate load sharing between roof, ceiling and floor diaphragms, and specifically designed bracing walls and "non-structural" walls. This paper gives a brief outline of the procedure used to test a complete house designed and built in accordance with current building regulations for houses in cyclone-prone areas. Hydraulically applied forces simulated the loads placed on the house during very high winds. The house response enabled weak points in the structure, and load transfer mechanisms, to be identified. The series of tests contributed to an increased understanding of house behaviour and will provide valuable feedback to the writers of current Building By-Laws.

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# 1. INTRODUCTION

The building regulations that ensure the safety of housing have evolved from building practices over many years. However widespread damage following cyclones 'Althea' in Townsville 1971 and 'Tracy' in Darwin 1974 showed that some of these practices produced structures that were unable to carry high wind loads. This experience together with the advent of new building techniques and materials has caused building regulations to be substantially rewritten without the benefit of decades of experience. Engineers have had increasing involvement in structural provisions for housing, but the complexity of the structural action of a house exceeds that of multistorey buildings. To circumvent a very complicated analysis, and to obtain a better understanding of housing behaviour in high winds, there has been much wind tunnel research (Holmes 1980) and laboratory testing of building elements over recent years. However, the interaction of all of these elements in complete houses still remained in conjecture, so a programme of full scale tests on real houses has been developed.

In 1982, a forty year old house was tested by applying simulated wind loads in both uplift and horizontal directions in a total of 8 tests with approximately 200 deflection readings being taken. In their report on this work Boughton and Reardon (1982) were able to draw conclusions on both the feasibility of testing full scale houses, and the action of houses in resisting wind loads. Those tests were particularly valuable for evaluating test analysis methods, as the house had been loaded by Cyclone Althea some ten years earlier with little or no damage. This gave a reference point for checking the test results. At the loads applied by Cyclone Althea the house should not have displayed much distress and that was indeed the case. However the small amount of data collected limited the conclusions that could be drawn from those tests. In order to rectify that problem an instrumentation system was built to enable direct recording of response data on a digital computer. An electrically operated hydraulic pump has also enabled a faster loading to be achieved and cyclic loading sequences to be used on the house reported in this work. Thus on the Hyne House, more than 40 times the data was collected and more than 8 times the number of tests were performed in about the same time taken to test the Garbutt House.

This publication outlines the test methods used on a new house built to current building standards and presents some preliminary findings. A complete set of recommendations will not be available until the detailed analysis of all of the collected data has been finalised. It will be presented in a later publication.

## 2. DESCRIPTION OF THE HOUSE

### 2.1 Design

The tests reported in this work were performed on a timber framed, high-set house with fibre cement external sheeting and largely sheeted with plasterboard on interior surfaces. The house was made possible by a generous donation by Hyne and Sons, timber merchants from Maryborough, Queensland. To celebrate their centenary they made available a sum of money that provided the impetus to start testing a new house built to current Building By-Laws. It is therefore generally referred to as the Hyne House and will be referred to as such throughout the rest of this publication.

The Hyne House was designed by the authors with the following aims in mind:

(i) The style of the house and floor plan were to be representative of contemporary high-set, three bedroom housing.

(ii) The structural details of the house were to comply with the provisions of the Home Building Code (1981), Appendix 4 to the Queensland Building By-Laws (1975), in respect of building in cyclone prone areas.

(iii) Where alternative details or construction methods were allowed in the Home Building Code, the authors intended to choose the one which was most likely to cause problems in the structural response of the house.

To achieve these aims the following design and construction technique was used. Prior to the commencement of the design of the house, eleven house plans of three bedroom high-set houses were collected from builders currently engaged in this type of work in North Queensland. A floor plan representative of these houses was then adopted. This included a large and open lounge/kitchen/dining area over which the roof and ceiling diaphragms would have to span the maximum distance permitted in the Home building Code - 6 metres. By way of contrast, the other end of the house featured many internal walls in the bathroom/toilet/bedroom area. Thus, if torsion of the house was to be a problem the imbalance in internal walls would highlight this effect. The bracing incorporated in the design used a combination of external fibre cement bracing panels and internal plasterboard walls. The bracing design was taken from the manufacturers' recommendations and gives the minimum resistance required by the Home Building Code. Bracing was accomplished by utilizing one 1200 mm wide fibre cement sheet on each wall at all four corners of the house as a bracing panel and two internal plasterboard clad walls as single sided bracing panels. One of these walls was 3300 mm long, and was designed with decorative random groove ply sheeting on the other face. The other was a 1200 mm long section of wall between the toilet and the bathroom with thin fibre cement sheeting on the other face. Cladding on all other external and internal walls was fastened according to normal building practice.

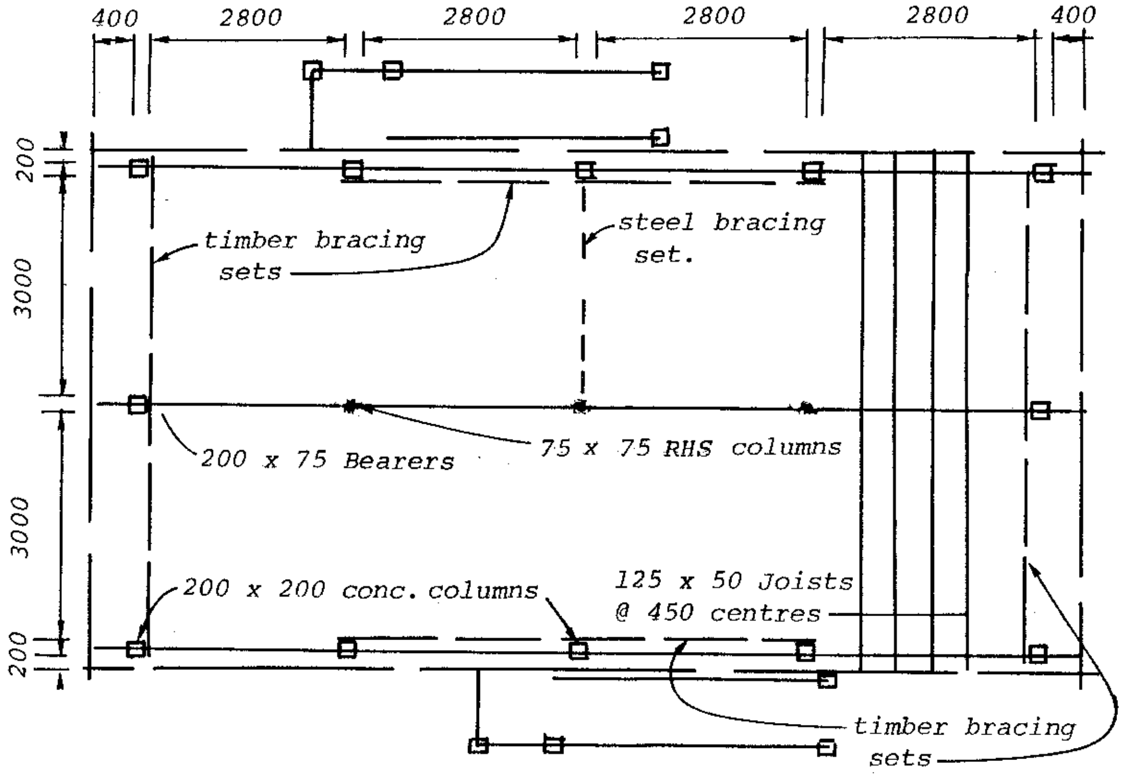
The roof was designed with a 10° pitch, as this gave very high uplift forces and yet still allowed room to move within the ceiling space to install displacement transducers and observe damage. The design also called for a trussed roof to enable assessment of the methods of tying ceiling diaphragms and the roof diaphragm action into the bracing walls. In the Home Building Code, it is required that a 12 mm minimum gap be left between the roof trusses and internal walls. This requirement calls for special details to transfer forces into the rest of the house. Custom Orb roof sheeting was chosen to give continuity with the laboratory studies on roof diaphragms (Nash and Boughton 1981) and (Boughton 1982). Roof tie-down mechanisms complied with the requirements of the Home Building Code and consisted of 10 mm bolts between roof batten and wall top plate adjacent to each truss, and 12 mm tie down rods between the wall top plate and the underside of the floor joints.

The Hyne House was designed to have a timber strip floor, as it was believed that the diaphragm action of most other types of flooring would have been superior. Each 60 mm board was nailed with one nail at each joist. The joists were skew nailed to the bearers, and steel framing anchors were used to effect tie-down at joists adjacent to the 12 mm steel tie-down rods. The bearers were tied into the top of the concrete columns with 6 mm steel flat plates, or bolted to the top of steel columns, with 10 mm bolts passing right through the depth of the bearers. Concrete columns were used around the periphery of the building and were designed as tie-down columns as defined in the Home Building Code. The central columns were steel RHS 75 x 75 mm and could not effectively be considered as tie-down columns as all tie-down forces from the trussed roof were taken by the 12 mm tie-down rods through the external walls to the concrete columns on the north and south side of the building. Thus fewer effective tie-down columns than the minimum number specified in the Home Building Code were used, but this was almost universal practice in North Queensland at the time the house was designed.

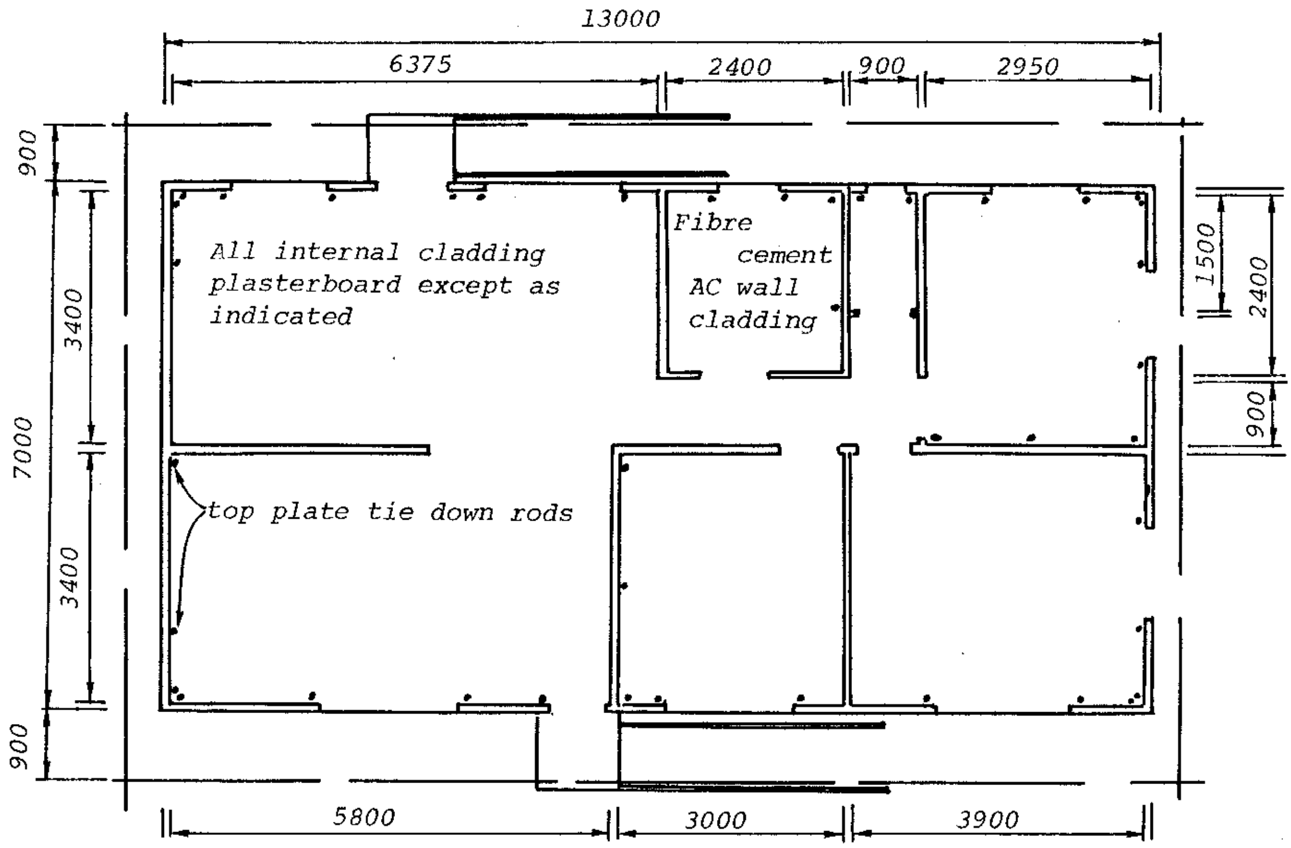
The lateral bracing below floor level was again according to the minimum provisions of the Home Building Code. Timber cross braces were provided at each end of the house, and a steel welded cross brace was installed in the centre of the building to give a comparison of the properties of the two types of bracing. Timber cross braces were also used on the north and south sides of the house to provide restraint to forces normal to the short direction of the house.

Drawings of the house are presented as Figures 1, 2 and 3, and were taken directly from the drawings provided in the specification for construction of the house. These drawings were checked by building inspectors of the Townsville City Council in accordance with usual building practice.

Again pains were taken to ensure that the construction of the house followed the usual practice for high-set houses, with quality of materials and workmanship typical of those encountered, yet still complying with the Home Building Code. Throughout the construction phase inspections were performed by City Council Inspectors, who not only ensured that the minimum standards of the Code were met, but also that work was typical of other local house construction. A contract for most of the work was let to a local house builder who ordered materials, constructed the subfloor, erected the prefabricated timber wall frames and roof trusses and applied external wall sheeting. The installation of internal wall cladding, ceiling sheeting and roofing was performed by tradesmen employed on a sub-contract basis.

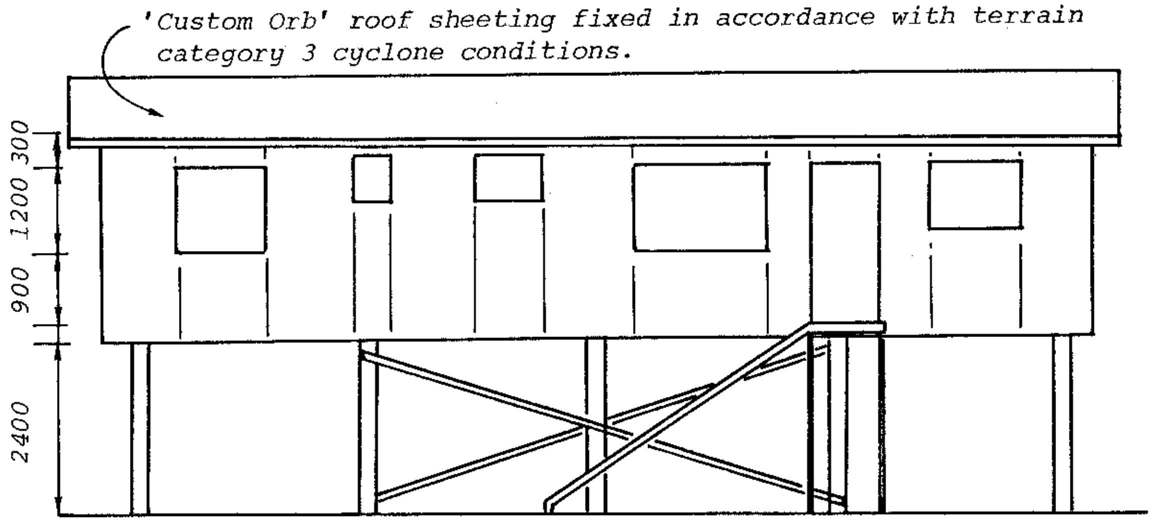



(a) Foundation Plan

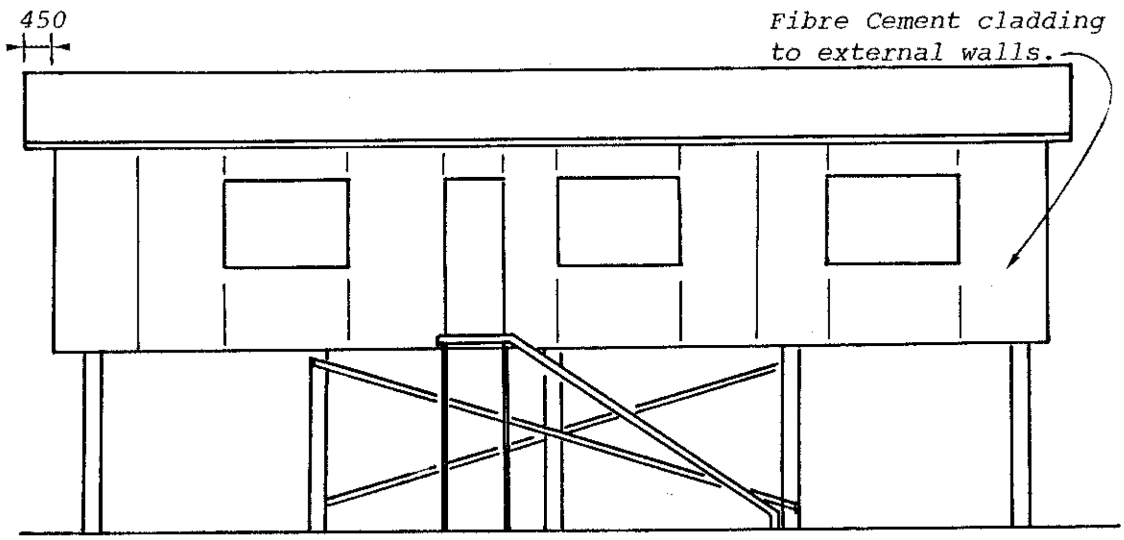



(b) Floor Plan

Figure 1 Plans of Hyne House

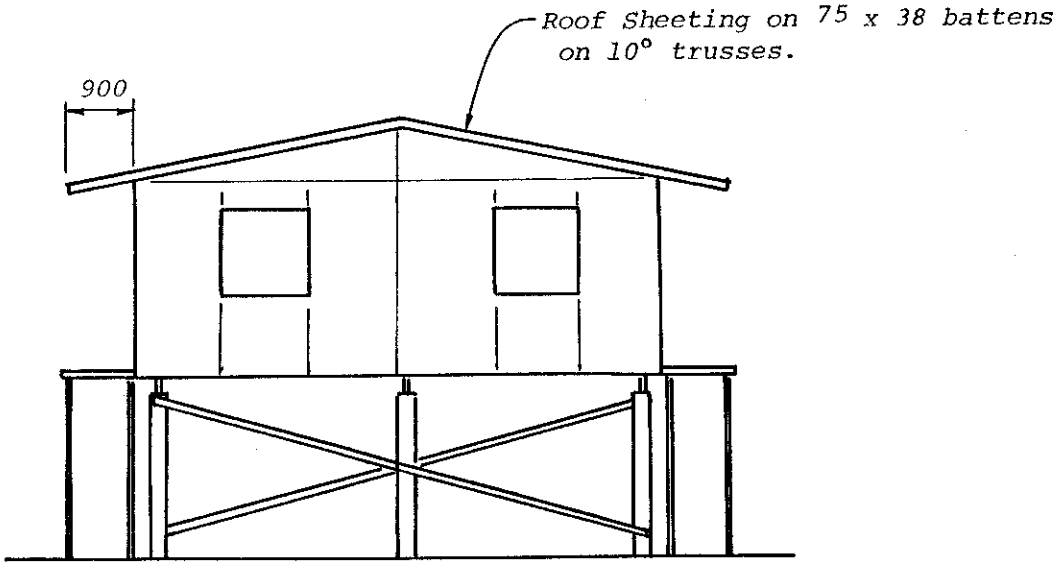



(a) Rear Elevation

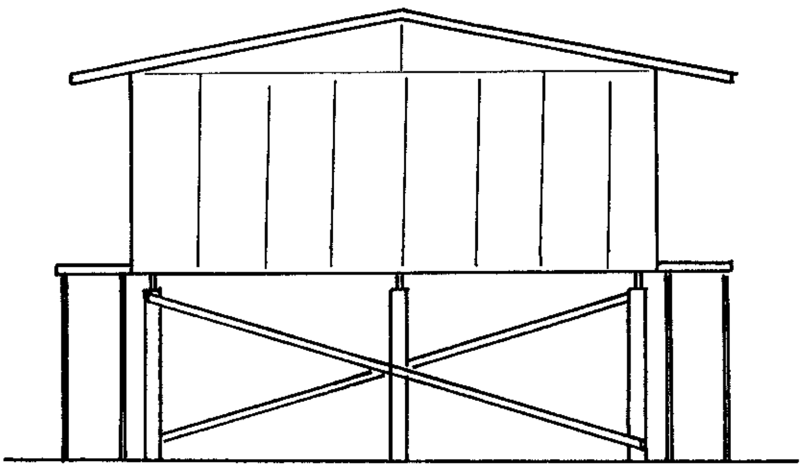



(b) Front Elevation

Figure 2 Elevations of Hyne House




(a) Side Elevation



(b) Side Elevation

Figure 3 Elevations of Hyne House

## 2.2. Construction Details

The 200 x 200 mm concrete columns were cast on site and the steel columns set into foundations. All subfloor bracing was completed prior to installation of the floor. The prefabricated hardwood exterior wall frames and softwood interior wall frames were nailed to the floor and supported with temporary props. 75 x 35 mm ribbon top plates were nailed to the existing top plates of the north and south wall frames. The ribbon plates ensured that there was clearance between the top of internal walls and the underside of the roof trusses. They were subsequently bolted to the top of the wall frames. The prefabricated roof trusses were then skew nailed to the wall top plate with two nails at each end. Hardwood roof battens were then screwed to the top chord of the trusses, and softwood ceiling battens nailed to the underside of the trusses. The ceiling battens were not continuous over internal walls and no ceiling nogging was used at walls parallel to the roof trusses. Where a wall ran perpendicular to the line of the roof trusses, a ceiling batten supported the top of the wall on each side, thus restraining it from falling over. Tie-down rods were then installed, and restraining rods were fitted in bracing walls. A 10 mm cuphead bolt was installed adjacent to each end of each truss and steel straps were used to tie the ceiling timbers into bracing walls.

At this stage the frame was inspected by Townsville City Council Building Inspectors. One inspection of approximately twenty minutes was made representing the usual time allotted to frame inspections. As a result of this inspection, nine deviations from the requirements of the Home Building Code were noted, of which the builder was required to remedy eight. The number of effective tie-down columns was insufficient as noted previously, but as this requirement is rarely enforced in North Queensland, the subfloor uplift restraint remained as built. Following the official frame inspection, five inspectors each spent half an hour examining the house to ascertain whether the quality of materials and workmanship were typical of local construction. Only two points were noted as being atypical. These were:

(i) The studs used in the prefabricated timber frames were all dressed to 70 x 45 mm whereas many frames incorporate 70 x 32 mm dressed studs where the fibre cement sheets do not lap. This was not considered a major concern for the following reasons. The studs were not loaded in bending as discussed in Section 4.1. As mentioned in Section 6.2, all walls were subjected to very high racking forces and neither the larger hardwood studs nor smaller softwood studs suffered damage that was a function of stud size.

(ii) The masonry anchors that secured the timber cross braces to the concrete piers were 20 mm diameter instead of the 16 mm specified by the Home Building Code. In fact 12 mm diameter fasteners are often used in conjunction with a pipe sleeve through the timber to achieve sufficient bearing area. The details around the masonry achors did prove critical in the evaluation of the subfloor bracing strength, and these will be further discussed in Section 6.3

After rectifying the eight items requiring further work, the internal and external wall claddings were installed by nailing in accordance with the manufacturer's recommendations for bracing and non-bracing walls. Plasterboard walls were filled at joints and nails but not at corners. The roof sheeting was then fixed and finally plasterboard ceiling sheeting installed. Again, the ceiling was finished flush.

A sumary of the construction details is given below.

| | |
|---|---|
| Perimeter columns | 12 only 200 x 200 mm reinforced concrete columns 2400 mm high and cast in place. |
| Central columns | 3 only 75 x 75 x 3.2 mm RHS steel columns. |
| Floor bearers | 200 x 75 mm F17 hardwood. |
| Floor joists | 125 x 50 mm F17 hardwood. |
| North-South pier bracing | 2 sets 100 x 50 mm timber cross braces fixed at each end and centre with a 20 mm expanding masonry anchor, 1 set steel 25 x 25 mm RHS cross braces welded each end to steel columns. |
| East-West pier bracing | 2 sets 100 x 50 mm timber cross braces fixed |

| | |
|---|---|
| | at each end and centre with a 20 mm expanding masonry anchor. |
| Flooring | 19 x 60 mm hardwood strip flooring fixed with one nail per board per joist. |
| Joist to bearer connection | 2 skew nails but also with two framing anchors connections adjacent to tie-down rods. |
| Bearer to column connections | (i) concrete columns - fish tailed plate 50 x 6 mm with one MI6 bolt.<br>(ii) steel columns - two M12 bolts through full depth of bearer. |
| External wall frames | Unseasoned hardwood, studs dressed to 70 x 45 mm, at 450 mm spacing. |
| Internal wall frames | Seasoned pine, studs 70 x 35 mm at 450 mm spacing. |
| Roof trusses | 7 m span, 10° pitch, 900 mm spacing, marked "Wind Design Velocity = 41.1 m/s". |
| Roof battens | Unseasoned hardwood, 75 x 38 mm at 840 mm spacing. |
| Ceiling battens | Seasoned pine, 45 x 35 mm at 450 mm spacing. |
| Roof batten to truss fixing | 1 only 14 x 75 mm steel countersunk head, Type 17 screw per crossover. |
| Roof sheeting | Lysaght Zincalume treated Custom Orb. |
| Roof sheeting fasteners | 14 x 50 mm Type 17 screws every second corrugation, but every corrugation along edges. |
| Truss to top plate tie-down | 1 only 10 mm cuphead bolt adjacent to each end of each truss. |

| | |
|---|---|
| Top plate to floor tie-down | 12 mm anchor rods from top plate to an under-batten under floor joists at each corner, adjacent to openings and at max. 1800 mm centres. |
| External wall cladding | Hardie's 4.5 mm external fibre cement sheeting. |
| Internal wall cladding | 10 mm thick plasterboard sheeting with recessed edges. |
| Ceiling cladding | 10 mm thick plasterboard sheeting with recessed edges. |
| Cornice | 100 mm wide curved plasterboard cornice. |

All claddings were fixed in accordance with manufacturers' recommendations.

While the house was structurally complete, it was quite unservicable, as it had no doors, windows, flashings, guttering, eaves lining, downpipes, stairtreads, plumbing, wiring, cupboards or fittings. All details not considered permanent reliable structural elements were omitted. This both saved costs in constructing the house and prevented a false picture of the house strength from being obtained. As doors can be left open, or blown open, and windows can be broken by airborne debris they cannot be considered as reliable structural elements in high winds.

The completed Hyne House consisted of a bare structural shell, typical in both design and construction of many three bedroom, high-set houses in cyclone-prone Queensland. Most importantly, it complied in every respect with the provisions of the current applicable building regulation - the Home Building Code.

## 3. WIND LOADS ON THE HOUSE

The Hyne House had an external geometry that was identical to one that was the subject of a wind pressure study by Holmes (1978). In this work, Holmes used a 1/50 scale model in a boundary layer wind tunnel where the wind velocity and turbulence experienced in a tropical cyclone were reproduced to the same scale as the model. 101 pressure taps in the model enabled surface pressures to be accurately measured and recorded by a digital computer for 18 mean wind directions. Following the analysis of this by Holmes the maximum total loads on the full scale house could be calculated for a wind speed of 42 m/s at eaves height. The report also enabled a wind direction to be chosen that would give the most adverse loads on the Hyne House, and gave insight into the way wind pressures are applied to houses.

Mean pressures on the outside of the roof surface were always negative - tending to lift the outer skin of the roof. The largest negative pressures occurred near the gable and the eaves on the windward side. The largest positive mean pressures were experienced on the windward wall and corresponded to wind directions essentially normal to that wall. The pressure on the underside of the eaves above the windward wall was essentially the same as that on the windward wall. The suctions on the leeward wall were generally small though significant.

Because of the large number of pressure measurement points examined, the total loads on the house could be calculated with a reasonable degree of certainty. The effect of pressure on each wall and roof panel could be resolved into a lateral force, an uplift force and an overturning moment about the centre of the floor.

These forces are listed in Table 1 and compared with similar forces calculated for the house using the Wind Loading Code (SAA, 1981). It is these latter forces that the provisions of the Home Building Code have been designed to resist.

The total lateral loads above were found by summing the pressure effect on the outside of the windward surfaces and the suction effects on the outside of the leeward surfaces. It was assumed that the internal pressures would remain fairly static and act with equal magnitude on windward and leeward walls, and hence would cancel. Internal pressure was therefore ignored in the analysis of the total lateral loads.

Similarly in evaluating the total overturning moment on the house, the internal pressures produce no net overturning moment so it was only the effect of external pressures that was significant.

Likewise the total uplift on the complete house was found using pressures and suctions on external surfaces including the underside of the floor. However as the loads were applied to the roof structure the effect of internal pressure was important. The uplift loads on the roof structure consisted of a suction on the outside of the roof surface and, in the event of windows and doors failing on the windward wall, a positive internal pressure. The effect of this high internal pressure gave much higher uplift loads on the roof structure than on the complete house, so the roof structure load was used in the tests

TABLE 1

| Calculated Design Loads on Hyne House for 42 m/s Wind Speed at Eaves Height | | |
|---|---|---|
| | Design Loads from Wind Tunnel Studies | Total Loads from AS 1170/2-1981 |
| Total Lateral Load | 41 kN | 43 kN |
| Total Overturning Moment (about centre of floor) | 101 kNm | 52 kNm |
| Total Uplift on Complete House | 68 kN | ? |
| Total Uplift on Roof Structure | 158 kN | 173 kN |

A design wind velocity of 42 m/s corresponds to the design value found using AS 1170/2-1981 for a house with a 6 m eaves height in suburban surroundings. It is also the wind velocity for which the provisions of the Home Building Code - the building standards to which the test house was designed and constructed - are applicable.

The total lateral and uplift loads obtained from the wind tunnel data differed from those calculated using the current code by less than 10%, which was reassuring. However the total overturning moment calculated directly from the wind tunnel data was twice that calculated from the code. Holmes suggests that this is due to the considerable overturning effect of the high uplift forces experienced at the leading edge of the roof. This is not reflected in the structural loads as derived from the code. In performing later analyses the design overturning moment as calculated from the wind tunnel data will be used as a basis for comparison with moments applied in the tests on the Hyne House.

The design loads are shown in Figure 4, plotted on an elevation of the Hyne House.

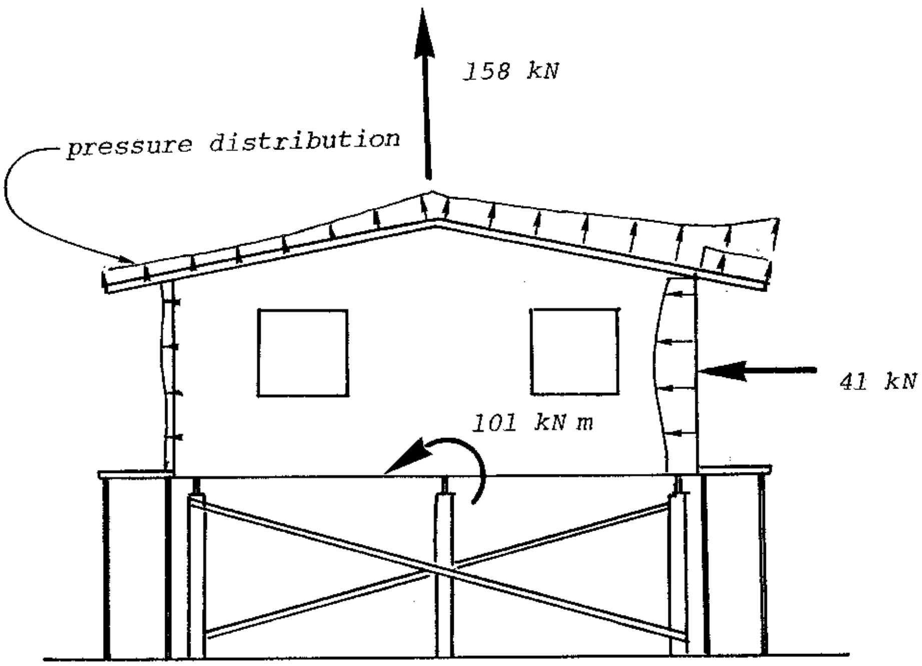



Figure 4 Design Wind Loads on Hyne House from Wind Tunnel Studies.

## 4. LOADING AND INSTRUMENTATION

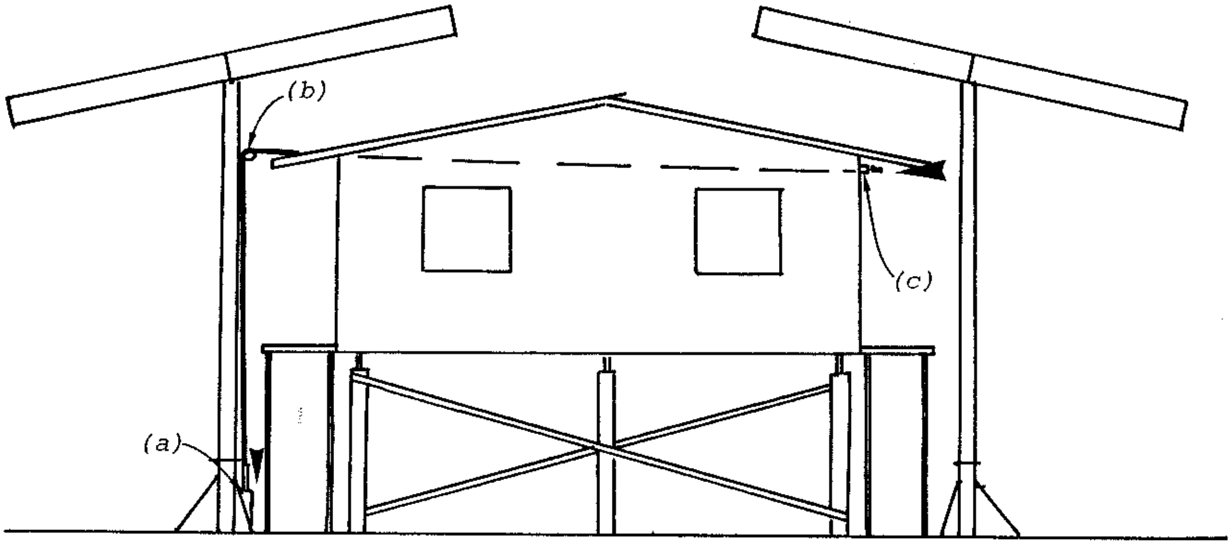
Having determined the 42 m/s design loads, forces were applied to the Hyne House to produce the same structural effect as those loads and deflection of the house was measured at many locations to obtain its structural response.

## 4.1 Loading System

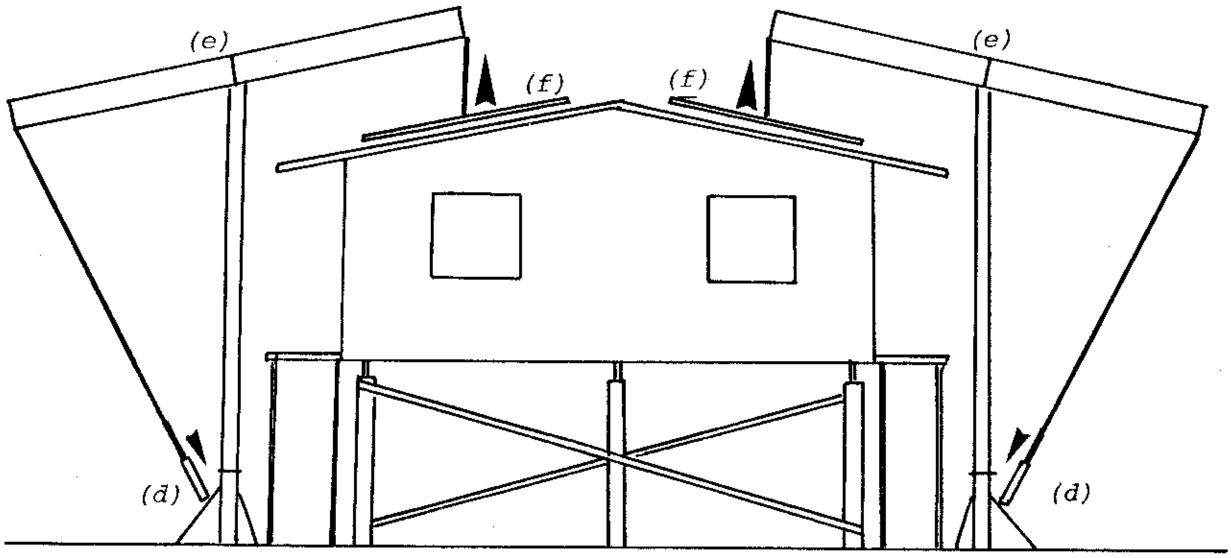
As was seen in the previous section, loads on the house could be divided into lateral loads and uplift loads. The lateral loads acting on the building consisted of pressure loads normal to the wall surface of the windward and leeward walls with the suctions on each side wall cancelling each other out. The action of the wall studs in bending, carries these loads to the wall top plate and the wall bottom plate. The mechanism of load transfer by the studs and the sheeting can be satisfactorily tested in the laboratory, so was omitted from this load simulation. Theoretical analyses, studies of damage following high wind events and tests performed on studs in a house by Boughton and Reardon (1982) indicate that there is ample stiffness and strength in the studs to transfer loads effectively to top and bottom wall plates as a uniformly distributed load with the same total value as the pressure loading on the whole of the house. A further simplification was made by applying the lateral loads to the windward side of the building only. In reality some lateral loads are caused by suction on the leeward wall, but the large number of trusses securely fixed to the top plates enables the windward and leeward walls to act together.

The lateral loads were therefore applied using the apparatus indicated in Figure 5 (i). The forces were applied by a hydraulic ram (a) which applied a tension force 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 spacing over the width of the house.

When the forces were equivalent to the 42 m/s design load, half of the total lateral force was applied at top plate level, as only the top half of the windward and leeward walls contributed to the force at that level. In separate tests when the load was applied at the bottom plate level, the total lateral force was used. This consisted of the pressure force from the bottom half of the windward and leeward walls transmitted downwards by the studs and the force from the top half of the walls carried up to the top plate, then into bracing and other walls, and down to floor level. Thus the bracing at and below floor level was subjected to the total lateral wind force, while that above floor level was loaded with half of the total lateral wind force.




(a) Lateral Loads




(b) Uplift Loads

Figure 5 Load Application System

The total design uplift forces acting on the house were taken as the highest forces of two alternatives as discussed in Section 3. The highest uplift forces were produced considering full internal pressure, and suction on the outside of the roof sheeting. Thus the appropriate place to apply the uplift loads was in the roof structure. It is generally accepted that with lined eaves, it is possible for the ceiling manhole to blow in and allow the full internal pressure to act on the underside of the roof sheeting. Therefore the total maximum uplift force can be applied directly to the roof sheeting. Again laboratory tests have been extensively used in the past to evaluate the performance of the roof sheeting, and the roof fastening systems. The roofing and fastener systems used in the Hyne House were of a type that have been found satisfactory in tests such as outlined by Reardon (1980). The uplift loads were therefore applied directly to the roof battens using a pair of load spreaders. The loading equipment utilised was as shown in Figure 5 (ii).

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 roof battens over a total area of approximately 20 $m^2$. This roof area incorporated 24 batten to rafter connections.

During all tests, forces were measured using strain gauge load cells. These had been previously calibrated in laboratory conditions, and for the first tests two cells were used in series. Thus both cells measured the same loads. This check ensured that the indicated loads were similar for both load cells. It also highlighted the need to protect both the load cell, and electronic readout equipment from direct sunlight, a precaution that was taken throughout the remainder of the test series. Two types of load cell readouts were utilised throughout the series. One produced a needle movement on a dial face and was used to control load increments, and the second interfaced the load cell with the same microcomputer that processed the deflection information. The latter method was used primarily during the cyclic loading tests.

## 4.2. Deflection Measurement

In order to record the response of the house to the applied loads, deflections were measured at up to 52 locations on the structure. A section of scaffolding on the front and back of the house that was independent of both the house and the loading system, provided the datum. Deflection measuring transducers were fixed to this datum with magnetic bases, and relayed information to a micro computer for processing and storage when required. The system is depicted in Figure 6 and described in detail by Boughton (1983).

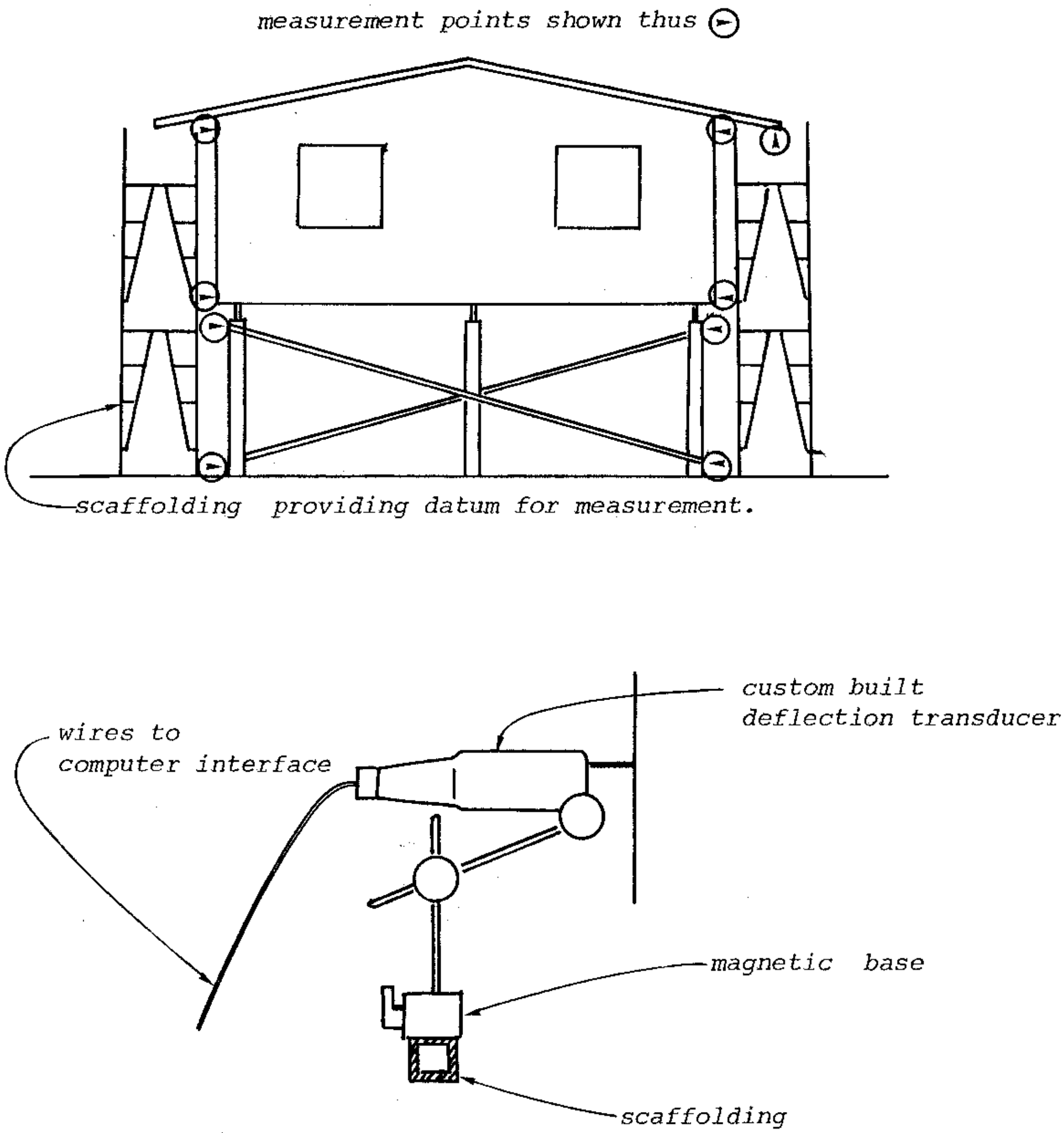



Figure 6 Deflection Measurement

During the lateral load tests, the transducers were positioned at approximately 1 m intervals along the loaded plate, at the top plate level and bottom plate level,at each wall that ran parallel to the loading cables, level, at and also at the top of each perimeter pile. All of these transducers recorded deflections relative to the ground as datum.

During the uplift tests however many of the deflections measured were local deflections within the roof structure. These included batten to top chord movement, movement across nailing plates in the trusses and movement of bottom chords of trusses relative to the top of internal walls. As well as these measurements, vertical deflection of the eaves and bottom plates were measured with the ground as datum.

The 52 transducers were read in less than 5 seconds which gave a typical test duration of 5 minutes including 10 load increments and producing over 500 deflection readings. The short duration of the loading gave little creep in most of the tests, and also enabled many tests to be performed in one day. In all, 66 tests were performed on the house, producing over 30,000 deflection measurements. These measurements were checked and printed by the computer during the test, but also stored on magnetic tape for later detailed analysis. Throughout the course of a test, load-deflection curves could be viewed on the computer screen. They verified that the house was behaving linearly during elastic testing, and enabled the commencement of failure to be pin pointed during destructive testing.

## 4.3 Cyclic Testing

As the house was designed for a cyclone-prone area and constructed in accordance with regulations established for housing in cyclone-prone locations, a series of cyclic load tests were performed. These tests were instituted to assess fatigue susceptibility or any tendency to degradation of the structure under repeated loading and were based on guidelines established in the Experimental Building Station's Technical Record 440 (1978).

The cyclic tests each followed a similar pattern with a prescribed number of cycles of zero to 5/8 design load, another set of zero to 3/4 design load, and finally a set of cycles from zero to design load. The pattern was implemented using the micro computer in conjunction with a hydraulic pump controller. At a signal from the computer the pump was started, and the loading cycle commenced. As the load was being applied, the computer continually monitored the load cell reading the force. When the prescribed maximum load was reached, the loading cycle was stopped, the deflections read if necessary and the unloading cycle commenced. Again at the prescribed minimum load the unloading cycle was stopped, deflections read if necessary and the next load cycle commenced. Most tests involved large numbers of cycles so deflections were read at selected times only during the test. Typically 20 sets of deflection readings were taken for each group of load cycles.

As the uplift loads were applied directly to the point of attachment of the sheeting to the battens, the following grouping was required:

| | |
|---|---|
| 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 on the house required a less severe loading sequence:

| | |
|---|---|
| Lateral loading | 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. |

## 5. ELASTIC TESTING OF THE HOUSE

An important objective of the tests on the Hyne House was the determination of the route through the structure by which wind forces are taken to ground. High winds exert pressures on external and internal claddings, and the resulting forces are transferred to the ground using complex interactions between the timber frame and the cladding fixed to it. While many tests have been performed on walls, ceilings or even roofing systems in isolation, few have been performed on a complete house.

Throughout the construction phase of the Hyne House, many tests were performed by loading parts of the house to less than their design load. For example, by loading each wall with in-plane lateral forces at the top of the wall, the stiffness of each wall as a bracing element could be found. If this was performed prior to ceiling or roofing installation there would be little interaction between the walls and the true bracing walls, and stiffnesses could be found. If the stiffness of one wall was for example 10 kN/mm displacement, and in a later test that same wall showed 0.5 mm displacement, then it could be said that that wall was carrying a bracing load of 5 kN. Thus by finding the stiffness of all potential bracing elements, the bracing forces in them during other tests can be calculated. Using this information, the path of the applied forces could be traced from the load application points to the foundations.

Forty six such tests were performed, with care being taken to ensure that no element was loaded to beyond its design load. Over 22,000 displacement readings were taken during those tests and are currently being analysed to produce the 'force map' of the house. The sequence of the elastic tests was as follows:

(i) After construction of all columns, floor bearers and floor joists, each set of columns was loaded horizontally at floor level, one set at a time. This established the stiffness of each set of columns which could be used later to relate the deflection at the top of each column to the force being resisted by it.

(ii) After the installation of the strip flooring, each set of columns was again loaded at floor level, one set at a time. Then a uniformly distributed load was applied to the floor. These tests enabled the assessment of the stiffness of the flooring, acting as a diaphragm.

(iii) After the wall frames, trusses, all battens and tie-down rods had been fitted, and all wall faces clad, horizontal point loads were applied at top plate level adjacent to each wall parallel to the minor axis of the house. This enabled the evaluation of the stiffness of every wall acting as a bracing element and again could be used later to find bracing forces in each wall from deflection measurements.

(iv) After the installation of the roof sheeting the horizontal point loads at top plate level were repeated. A uniformly distributed lateral load was applied to the top plate as a separate test. This enabled the evaluation of the stiffness of the roof sheeting acting as a diaphragm.

(v) The roof sheeting was then removed, and after installation of the ceiling cladding, the horizontal loads at top plate level were repeated. This provided stiffness information on the ceiling cladding, acting as a diaphragm.

(vi) After the roof sheeting had been replaced, the horizontal loads were again repeated to give the stiffness of the combined roof sheeting and ceiling diaphragms.

(vii) Finally after the cornices had been installed, the horizontal point and uniformly distributed loadings were again repeated. This enabled the evaluation of stiffness information for the completed house.

During each test, load-deflection curves were observed to ensure that elements were behaving elastically. The elastic properties of the vertical bracing elements such as sub-floor columns and internal and external timber framed walls could be used to evaluate the performance of horizontal diaphragms such as roof sheeting, ceiling and floor boards.

The results of these tests will be published in detail at a later date, but they do indicate the following:

(i) Roof and ceiling panels do function as stiff diaphragms to transmit lateral pressure forces on walls to vertical bracing elements.

(ii) The strip flooring also functions as an effective diaphragm to transmit lateral loads from the base of walls to the braced pier sets.

(iii) All walls function effectively, within their elastic range, to carry lateral loads from top plate level to floor level. The presence of small windows in these walls does not greatly reduce the wall stiffnesses.

These preliminary findings will be quantified and expanded in later work and will enable the evaluation of total house performance when subjected to high wind or other large loads.

## 6. DESTRUCTIVE TESTING PROGRAMME

After construction of the Hyne House was completed, it was subjected to a series of destructive tests to ascertain failure loads of the structure and pinpoint any structural weaknesses should they have been present. The tests followed similar patterns. Loads were applied in a cyclic manner as described previously in Section 4.3. These tests were designed to highlight any parts of the house that deteriorated under fatigue loadings. After the completion of the cyclic load programmes, the house was subjected to a steadily increasing load until failure occurred. In many cases the failure was preceded by a yielding of material and an effort was made to find the points at which failure was initiated.

These tests were performed in the following order to maximise the information gained from the destructive test programme.

1. Lateral loads on the above-floor section of house to 4.75 times design load.

2. Uplift tests on 20 $m^2$ of roof structure.

3. Lateral loads on the above-floor section of house after simulated debris damage had been sustained on the end walls.

4. Lateral loads applied near the top of selected potential bracing walls.

5. Lateral loads on the sub-floor section of house, with the house as originally constructed.

6. Lateral loads on the sub-floor section of house with modified column braces fitted.

Rather than discuss the findings of these tests taken in chronological order, which could lead to confusion, they will be discussed under three basic headings: uplift on roof, lateral loads at top plate level and lateral loads at floor level.

## 6.1 Uplift on Roof

Uplift loads were generated and applied as indicated in Section 4. The cyclic loading sequence of 10200 cycles as detailed in Section 4.3 was applied first, and then the uplift load was steadily and slowly increased until failure first occurred.

During the first series of cyclic loads, 8000 cycles at 5/8 design load it was noted that one batten screw was broken. It was assumed that this took place during the installation of the screws as the fracture did not appear fresh. This screw was in a particularly critical location, immediately adjacent to a load application point and at a joint in the batten. Thus the load application point was 800 mm away from the nearest support causing the batten to act as a cantilever. As the test load was applied at a series of points rather than as a pressure loading, the cantilever action greatly exaggerated the bending moment in the batten. Thus if the loss of the batten screw caused the batten to fail in bending during the test, it would have been not a valid representation of its behaviour under real wind loads. The broken fastener was therefore replaced with a framing anchor prior to the application of the 200 design load cycles.

Throughout the application of the cyclic uplift loads, it was noted that all loaded trusses moved a few millimetres away from the wall top plates. This was due to progressive pull out of the two skew nails between the trusses and the top plate with load being shed to the 10 mm coach bolts countersunk into the batten and connected to the underside of the top plate. The extra load carried by these bolts caused further embedding of the head of the coach bolt into the hand cut recess in the batten. This allowed the one or two millimetre permanent upward movement of the trusses without causing any loss of strength of the tie-down system. No other permanent damage to the roof structure or tie-down system was noted. The 10 mm midspan deflection of the bottom chord of the trusses, measured during the design load cycles, proved to be elastic. This movement did cause some damage to cornices.

During the steady increase of load to failure, a very sudden movement took place when six batten screws pulled out of the top chord of one truss. The average load in the fasteners at this point was 4.6 kN per screw or 3.3 times the design load. As not every fastener in each loaded truss experienced this load, the uplift load on each truss and the tie-down system as a whole was only 2.2 times the design load. The tie-down system was not showing any signs of permanent damage, but the trusses on each side of the one that had lost the batten screws each showed some permanent damage. The steel toothed plates that secured the mid point of the top chord to a brace had pulled out of the brace on one side of these trusses. By examining load-deflection curves for those trusses it was possible to deduce that these truss failures occurred as a result of load redistribution following the screw failures. Laboratory tests have shown that failures of nails and nailing plates in shear have fairly ductile characteristics with a large amount of movement prior to failure. However, the load-deflection curves obtained for these trusses during the house test showed linear behaviour right up to the failure point. This suggests that the loss of the six screws on the adjacent truss caused load redistribution to the trusses under examination. The extra load would have been transferred by eight battens causing fastener loads just less than the 4.6 kN per screw required to cause pull out. But for these trusses the loads now applied were also approximately 3.3 times the design load rather than the 2.2 times design load or less that the others were carrying. The joint under examination was able to support 2.2 times the design load on the truss, but not 3.3 times the design load. That is, failure load for the truss joint was between 2.2 and 3.3 times the design load.

The 4.6 kN per screw failure load for the batten screws was less than the average obtained by Reardon (1979), but it was noted that the top chords of the trusses had split noticably during the installation of these screws.

The splitting may have contributed to a lower failure load, although some of the samples tested by Reardon also showed signs of splitting. However, in the house test configuration the six screws on each truss were being loaded simultaneously. Thus if the weakest joint failed at 4.6 kN per screw, the load removed from it would have been redistributed over the remaining five on that truss, giving an average load per screw of 5.5 kN. This load was the average failure load obtained by Reardon for these fasteners, so it is not surprising that the remaining five fasteners also failed. It is also to be noted that of 14 tests performed by Reardon, the two lowest values obtained were 4.6 and 4.7 kN per screw, with splitting of the rafter evident. This supports the failure load obtained in the house tests.

Thus minimum factors of safety obtained in the test were as follows:

| | |
|---|---|
| Batten screws | 3.3 |
| Truss | >2.2 |
| Tie-down system | >2.2 |

As previously mentioned, the roof sheeting and fastening system employed on the Hyne House was one that has been found to have factors of safety of more than 2.0 in equivalent laboratory tests. It therefore appears that all elements in the uplift forces chain of resistance have minimum factors of safety as built of at least 2, and should therefore perform satisfactorily under design wind conditions.

In subsequent tests the 10 mm cuphead bolts securing each truss were removed, and only two nails used to hold down the trusses. Under this configuration, the trusses lifted at less than design load, illustrating the effectiveness of the 10 mm bolts.

The roof structure as designed and constructed in accordance with the Home Building Code showed adequate strength to resist design uplift loads with satisfactory factors of safety.

## 6.2 Lateral Loads at Top Plate Level

Again, these loads were generated and applied as indicated in Section 4, with a cyclic loading sequence of 1020 cycles as detailed in Section 4.3 followed by a series of steadily increasing loads specifically to cause failure.

The cyclic loading sequence proceeded without incident, and the whole house behaved elastically with load and unload portions of the cycles following much the same load-deflection curve. No sign of damage was visible at the conclusion of the tests.

However, the steadily increasing loads to failure caused numerous problems due to the poor performance of the sub-floor bracing. The bracing was strengthened four times throughout the course of these tests, but even so the total lateral load was limited to 97 kN, equivalent to 4.75 times design load at top plate level. This limit was placed to prevent failure of sub-floor structural details which could have led to undesired severe damage to the house and testing equipment. The actual sub-floor bracing problems will be discussed in Section 6.3, and all comments in the remainder of Section 6.2 refer only to the house above floor level.

The house, as built, was able to sustain 97 kN uniformly distributed lateral load at top plate level. This load was equivalent to 4.75 times design load for 42 m/s wind speed at eaves height or 2.0 times design load for 65 m/s. At this load there was some deformation of doorways and windows, opening of some cracks between the cornice and internal walls, and some signs of distress near fasteners in plasterboard clad walls. The load-deflection curves were quite linear and indicated that the ultimate lateral load of the house as a whole could have been considerably higher. There seemed little point in loading the structure with forces that were higher than those equivalent to twice terrain category 1 design loads, so the effects of variation in construction or other damage were investigated. The house was realistically modified to simulate other types of construction or wind damage to claddings associated with air-borne debris.

(i) The roof sheeting was unscrewed, so that it could not effectively function as a diaphragm, however the weight of the overall roof structure was unchanged. This configuration gave much the same resistance to lateral loads as a tiled roof which has no continuous diaphragm membrane, but a large weight to mobilise some friction forces. The house was again able to sustain 4.75 times design load for terrain category 3 loadings without sign of serious damage, and showed a behaviour which differed only marginally from its behaviour in the previous test.

(ii) The roof sheeting was then replaced, and the uplift tests detailed in Section 6.1 were performed, resulting in minor damage to the roof structure. Less than 5% of the roof area was affected by this damage so the roof diaphragm was largely intact. During the previous tests it was noted that the cornice provided a very strong and direct link between the ceiling and the potential bracing walls. By directly connecting the ceiling cladding to the wall sheeting, shear forces could travel from one diaphragm to another bypassing fasteners in both the ceiling and the walls, timber framework and special bracing wall connections. The cornice was completely removed from the house. This was equivalent to a small timber strip cornice nailed only to the wall frame, and also confirmed that the cornice had carried most of the shear forces into the walls, as the top row of fasteners in the wall plasterboard which had previously been concealed behind the cornice were showing no sign of having carried load. The fasteners near the bottom of most plasterboard walls, however, were showing signs of having carried very significant shear forces out of the walls to the floor. Again 4.75 times the terrain category 3 design load was applied to the house without major additional damage. The fasteners at the top of plasterboard walls did show signs of having carried load from the frame into the wall bracing panels. While the roof and ceiling behaved as very effective diaphragms there was no sign of tearing at either roof or ceiling fasteners.

(iii) A very significant form of damage in cyclones is debris damage. Windborne pieces of vegetation or building materials can pierce the cladding on walls which are later required to act as bracing walls after a wind direction shift of 90°. The effect of debris damage was simulated on the Hyne House by damaging first one end wall of the house. Pieces of timber were thrown at the fibre cement sheeting to puncture each sheet in at least one place and remove less than 5% of the actual sheeted area. This was considered equivalent to moderate debris damage in tropical cyclones. The house in this condition behaved in a manner that was indistinguishable from its behaviour in the previous test, with no further cracking of the sheeting and no increase in the obvious damage to the house.

(iv) The very high loads that the house was successfully able to withstand suggested that very severe debris damage could be expected to accompany such high velocity winds. Thus both end walls were subjected to a heavy bombardment with pieces of timber. Approximately 50% of the external sheeting was physically removed from the end walls, the internal sheeting was punctured in about ten places on each end and the plasterboard had been pushed over the heads of some fasteners by the continual pounding. Again 4.75 times design load was applied to the house but this time some damage occurred during the course of the tests. The damaged end walls deflected more than in previous tests, accompanied by buckling of the plasterboard sheeting where it had been pushed over nail heads, and cracking of the remaining external fibre-cement sheeting. The cracks in the external sheeting originated at holes caused by the debris damage and moved upwards towards the loading points and downwards away from the load points. They were thus oriented across the tension diagonals in the bracing wall. See Figure 7 for a sketch of this damage. As these external walls were not capable of carrying the same loads that they had in previous tests, more load was transferred to internal bracing walls. After the total load of 97 kN had been attained, an internal bracing wall failed by tearing of the fasteners in the plasterboard sheeting and a buckling of the sheeting itself. Following this failure, the house carried 77 kN, equivalent to 3.8 times design load inspite of the fact that the main internal bracing wall had failed and that both external bracing walls had been severely damaged. The load was carried primarily by a 1.2 m long bracing wall and four other walls not designed specifically as bracing walls.

It is also to be noted that prior to the commencement of this test, the house had been subjected to four tests which achieved a load of 4.75 times design load as well as a cyclic load series incorporating 1020 load, unload cycles. It had experienced minor damage but only 4 mm of creep prior to the test.

(v) Next the entire roof structure was separated from the walls, lifted 15 mm and supported on strips of plasterboard to provide a sliding surface. This lessened the effectiveness of both roof and ceiling diaphragms and allowed individual walls to be tested to failure. These tests showed that even walls not designed as bracing walls were capable of resisting quite high lateral loads. A detailed report of this series of tests will follow in a later publication.

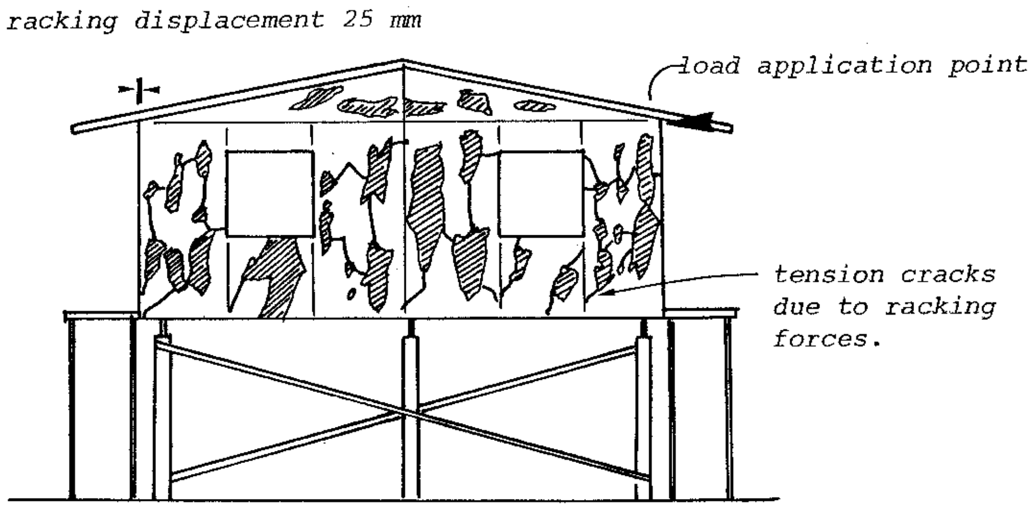



Figure 7 Debris Damage to Side Walls

## 6.3 Lateral Loads at Floor Level

The lateral loads at floor level were designed specifically to test the ultimate performance of the sub-floor bracing. Problems were associated with the testing of this part of the house as alluded to in the previous section. In the course of loading the house at top plate, those lateral loads had in turn been passed through the structure to the sub-floor bracing. Thus the cyclic loading of the top plate of the house had also caused cyclic loading of the sub-floor bracing although the forces were only half of the appropriate ones for the sub-floor system. However, in the course of applying failure loads to the top plate, the sub-floor bracing was loaded to in excess of design load fourteen times. In the course of applying these loads damage was caused to the foundations and repair and strengthening effected. A summary of these events is included below. Later detailed analysis may show that much is to be learnt from these failures also. An annotated plan of the footings is shown in Figure 8 with some sketches of the damage incurred.

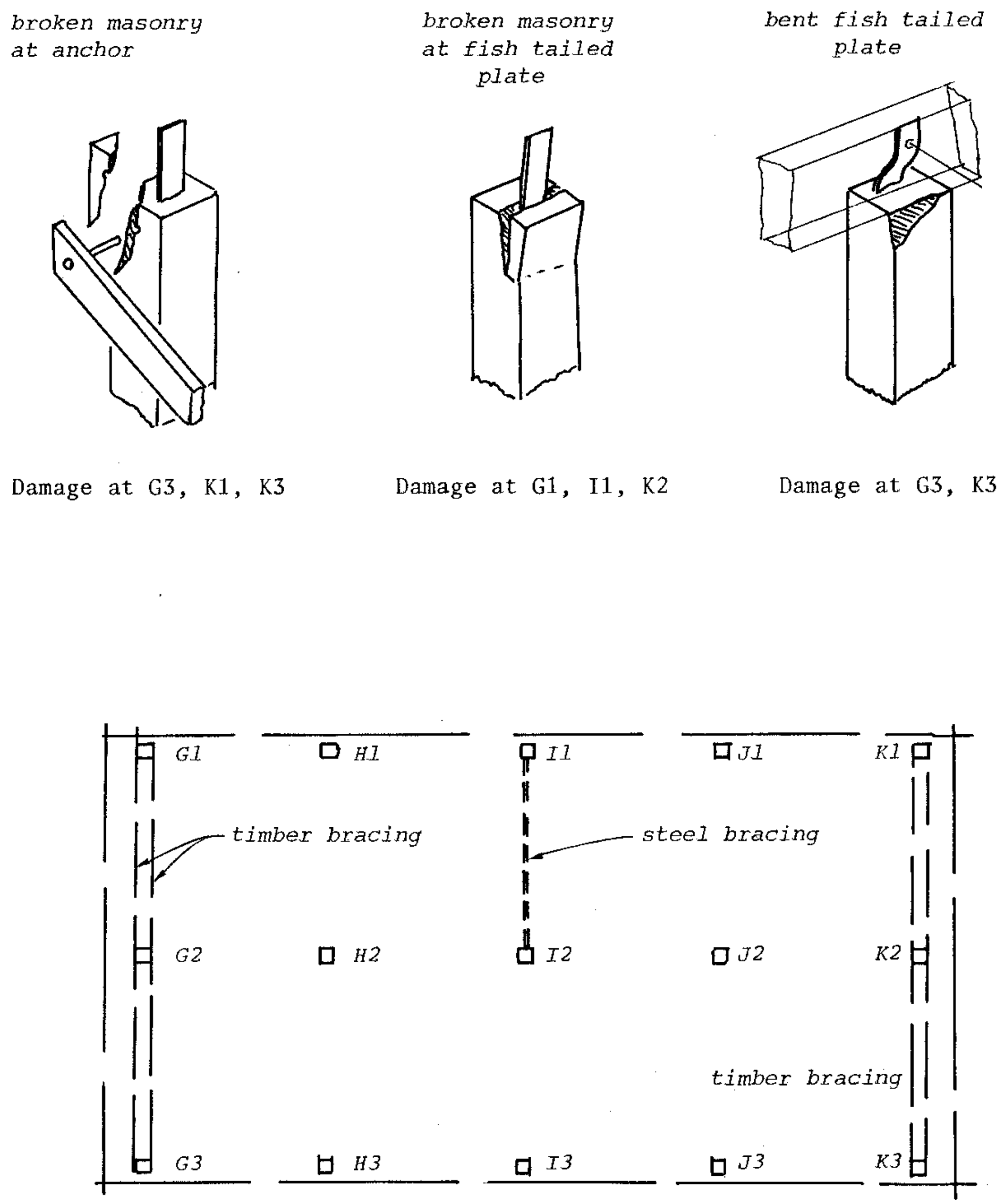



Figure 8 Footing Damage

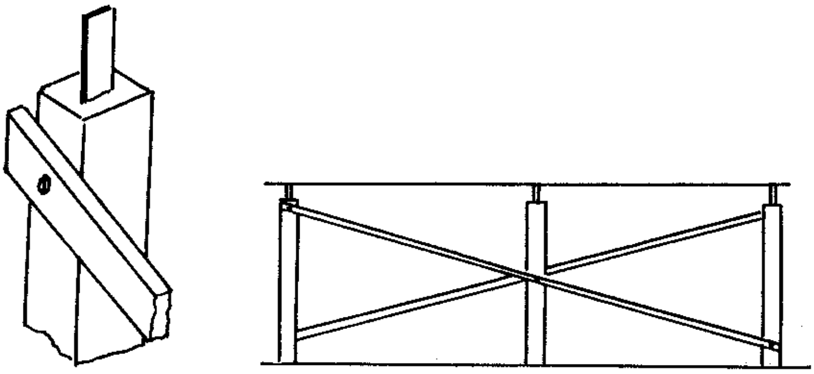
(i) During the application of a uniformly distributed load at top plate level, failures occurred at column K1, and K3 when the total load on the house was 59 kN or 1.4 times the design load for the sub-floor bracing system. It could be deduced from the load-deflection curves that the fish-tailed plate that secured the bearer to column K3 failed in bending first which removed load from the tension diagonal of the timber bracing set on columns K1, K2 and K3. This transferred extra load to the compression diagonal and caused the concrete failure at the top of column K1.

(ii) To prevent a recurrence of that type of damage, temporary steel diagonal tension braces were installed between the top of column K2 to the bottom of column K1. They were secured at each end by two 12 mm expanding masonry anchors into the face of the concrete. The bearer immediately over the column was prevented from rolling by the use of metal brackets as shown in Figure 9. This was repeated for the columns in set G, and similar braces were installed in sets H and J, but the top of the braces were welded to the steel columns.

(iii) During the application of a uniformly distributed load at top plate level a failure occurred at the top of column K2, again associated with the fish tailed plate securing the bearer to the column. The total load on the house was again 59 kN or 1.4 times design load on the sub-floor bracing.

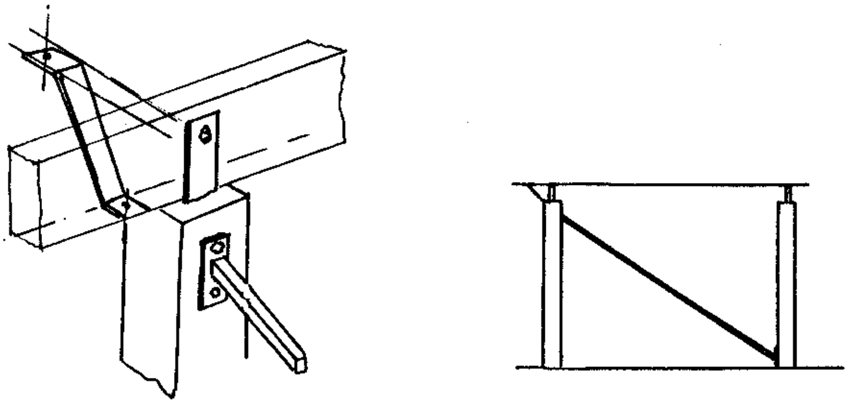
(iv) To remove the top of the concrete column from the line of force transfer from house to ground, the steel braces were extended and bolted directly to the top of the bearer over columns G2 and K2. Thus forces from the floor could be carried through the joist to the central bearer, then directly to the steel tension brace and to ground. Framing anchors were used to secure joists to the top of the central bearer thus ensuring efficient lateral force transfer from the floor to the bracing.

(v) At a total lateral load of 93 kN, the expanding masonry anchors securing the base of the steel tension braces to the inside face of the concrete columns failed by pulling out on three separate occasions. The expanding masonry anchors were replaced with 12 mm



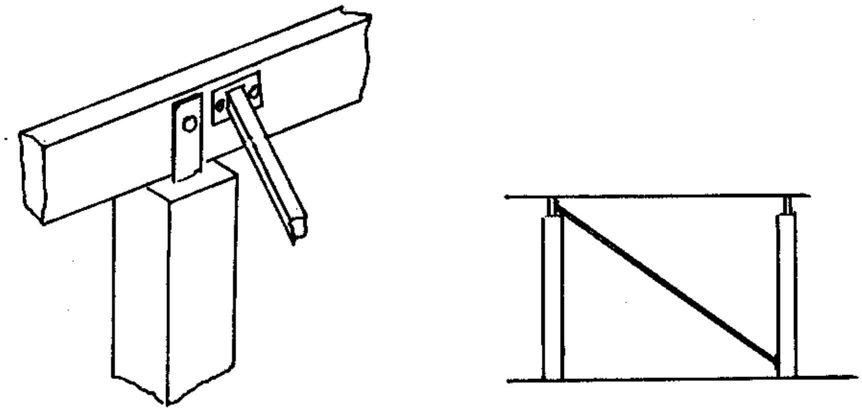
*timber braces fixed with one 16 mm masonry anchor each end*

(a) Originally installed



*32 mm steel RHS fixed with two 12 mm masonry anchors each end. Bearer restrained against rolling.*

(b) First Modification



*32 mm steel RHS fixed to top of bearer with two M12 Bolts. Base fixed with two 12 mm chemical anchors.*

(c) Final Modification

Figure 9 Column Bracing Systems

chemical anchors and the bracing system performed well for the remainder of the above floor tests.

At the completion of the tests on the house above floor level, all of the abovementioned temporary bracing was removed. Where concrete had been broken, it was replaced with high strength concrete grout, and the timber braces on columns K1, K2 and K3 changed to the other face of the column where the concrete was undamaged. This also enabled the masonry anchors designated as the minimum allowable in the Home Building Code to be used. The framing anchors connecting floor joists to bearers that had been installed as detailed above were broken to allow any weakness in that joint to be highlighted if it existed.

The house was then loaded by applying a uniformly distributed lateral load to the floor at joist level. At a total applied load of 62 kN, equivalent to 1.5 times the design load on the sub-floor bracing, the masonry anchor securing the top of the tension brace to column K3 failed by pulling sideways out of the concrete, however load did not drop appreciably. At 69 kN, the fish-tailed plates at the top of columns G1, and I1 and K1 started to bend, spalling concrete to expose the reinforcing cage. This was a slow process, and again load did not fall dramatically as failure progressed. However at 70 kN, equivalent to 1.7 times design load, the masonry anchor securing the top of the timber compression brace to column K1 failed by pulling sideways out of the concrete. Load did fall off as a result of this failure. At this point bending of the unbraced columns was quite noticable and the central bearer had rolled over column I2 so much that the top of the bearer was displaced 80 mm sideways relative to the bottom of the bearer. The compression members of the cross brace sets had buckled severely and were showing over 150 mm central sideways deflection. Finally at 74 kN, 1.8 times the design load, the masonry anchor securing the top of the timber tension brace to column G3 failed by breaking sideways out of the concrete. While the previous failures of this type had broken the concrete outside the reinforcing cage, this time the break passed through the cage so that the broken out concrete was held in place by the steel.

The generally low safety factors obtained in this series of tests may be of some concern, and further work should be carried out to ascertain any shortcomings of the sub-floor bracing system incorporated in the

Hyne House. A load factor of 2 is generally recommended for design of domestic structural elements for high winds, and with the first failures occuring at only 1.4 times design load there is a need for reassessment of the provisions for timber cross bracing. The steel cross brace between columns I1 and I2 with chemical masonry anchors at the base of the tension diagonal performed very well and towards the end of the test must have been carrying much of the total load applied.

The failures of the sub-floor bracing system were all associated with local tension zones in concrete columns which were not designed for such stesses. The obvious solutions for such a problem are either to design for such stresses by including special reinforcing in the column at the brace connection, or to avoid the tension zone by connecting the diagonal brace to some other member. The latter solution was the only one available to try on the test house. Therefore 32 mm square RHS steel diagonal braces were installed from the bottom of the concrete columns to the top of the timber bearer. The braces were bolted with chemical anchors to the column and with two M12 bolts to the central bearer. Figure 9 shows details of the brace.

The braces were installed on the end columns only, as they were incorporated into an investigation to assess the performance of the strip flooring as a diaphragm. The steel cross bracing between columns I1 and I2 was removed, causing the flooring to span 14 metres between the end braces. This system adequately resisted two times design load, thus demonstrating that strip flooring can act effectively as a diaphragm, even over a 14 m span. The test also showed that the experimental diagonal braces performed adequately.

## 7. CONCLUSIONS

The tests on the Hyne House have shown that there is great value in testing complete houses by simulating high wind loads. Analysis of the results can find mechanisms of load transfer, can demonstrate load sharing between the elements in the house and pinpoint areas of weakness or excessive strength.

In particular for the high-set, timber framed Hyne House:

(i) The roof, ceiling and floor all functioned as highly effective diaphragms to transmit lateral forces to the top of vertical bracing elements.

(ii) All walls within the house functioned well within their elastic range at working loads, to carry lateral loads from top plate level to floor level.

(iii) The roof structure and tie-down system functioned adequately to give a minimum load factor at failure of 2.2.

(iv) The house structure above floor level had great reserves of strength to resist lateral racking loads inspite of severe debris damage to external load carrying walls. The load factor of 4.75 at failure indicates sufficient strength to safely resist 65 m/s winds.

(v) The sub-floor bracing system as originally installed has some shortcomings and therefore needs further research. However, with adequate bracing, the combination of floor diaphragm action and end braces only has sufficient strength to resist the lateral loads with a load factor greater than 2.0.

These findings have been released prior to a detailed structural analysis of several parts of the test programme. Further investigatory work by the authors will produce additional conclusions and recommendations which will be published in 1984.

## 8. ACKNOWLEDGMENTS

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| | |
|---|---|
| Australian Gypsum Limited<br>CSR Limited | plasterboard for all internal walls and ceilings. |
| James Hardie & Coy Pty Ltd | fibre cement sheeting for external wall cladding. |

| | |
|---|---|
| Lysaght Brownbuilt Industries | roof sheeting. |
| Spurways Cooke | screws and nails. |
| Teco Pty Ltd | framing anchors. |

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