JAMES COOK CYCLONE STRUCTURAL TESTING STATION

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SIMULATED WIND TESTS ON A HOUSE

Part 2 — Results

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SIMULATED WIND TESTS ON A HOUSE

Part 2 Results

G.N. Boughton*

SUMMARY

Loads in excess of those experienced in tropical cyclones were placed on a 40 year old timber framed duplex in a series of tests designed to trace force paths through the building and determine the strength of the building's components. In this report the loads applied to the building in cyclone 'Althea' and those which would be applied in the current code design high wind event were calculated. These loads were compared with the failure loads obtained in the tests and conclusions drawn as to the strength of the building. These conclusions have highlighted the need for the improvement in building regulations that has taken place since construction of this house, and as detailed in modern building by laws.

An analysis of the lateral load distribution through the house has identified the structural role of the ceiling and roof sheeting, as well as internal walls.

* Research Fellow, Cyclone Structural Testing Station

SIMULATED WIND TESTS ON A HOUSE

Part 2 Results

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1. INTRODUCTION

In nearly all facets of industry, it is necessary to conduct tests on full scale prototypes to determine product response under known loading conditions. This is certainly true of the housing industry, as failure to withstand wind loadings can produce catastrophic results as was evidenced in Cyclone 'Tracy' in Darwin 1974 (Walker, 1975).

The Cyclone Testing Station's house testing project meets the need to test full-scale houses subjected to high wind loads. The project has been outlined in Boughton and Reardon (1982) and this publication presents the results of the first series of tests. The tests were of two basic types: strength tests in which the main object was to determine the load at failure of the structural elements subjected to the test, and stiffness tests in which local and large scale damage was specifically avoided during the test in an effort to determine force paths within the structure. In this way, the mechanism by which the house resisted working loads can be deduced from the stiffness tests and points of weakness in the structure identified using the strength tests.

The house used in the tests was 40 years old, so this report does not claim to be a comment on the strength of houses in general. Rather it serves to illustrate the action of timber framed housing in resisting wind loads and provides a means of evaluating the worth of the house testing project. Nevertheless the results of this series of tests serve to reinforce the need for comprehensive building codes with resistance of high winds as a major consideration.

2. DESCRIPTION OF HOUSE AND TESTS PERFORMED

Descriptive information was presented in detail in Boughton and Reardon (1982) but will be summarised below.

A timber framed duplex was supplied for destructive testing to the Cyclone Testing Station free of charge by the Queensland Housing Commission. The building was structurally sound but had been condemned for sanitary reasons. It was constructed during the Second World War, and was used by the United States Air Force as an unlined building. At the end of the War, it was taken over by the Queensland Housing Commission, fitted with internal walls, ceiling and wall linings, and converted to a duplex. Over its 40 year history, the house had been subjected to high wind loads from three major cyclones including cyclone 'Althea' which damaged many other buildings throughout Townsville. However, as all the sheets of roofing and weatherboards on the house were the originally installed items, the house appeared to suffer minimal damage in these three high wind events. A sketch of the home is shown in Figure 1, and some relevant structural details are summarised below.

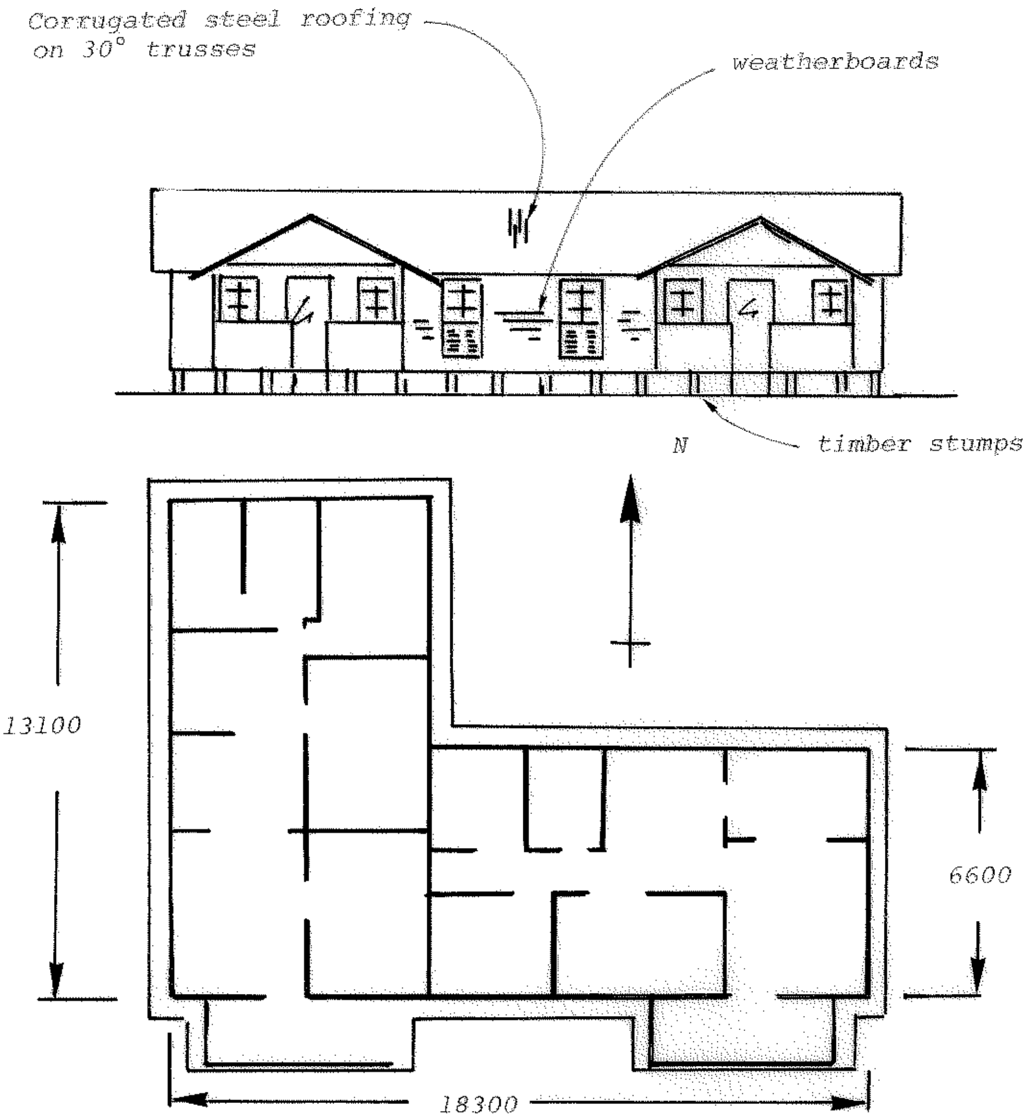
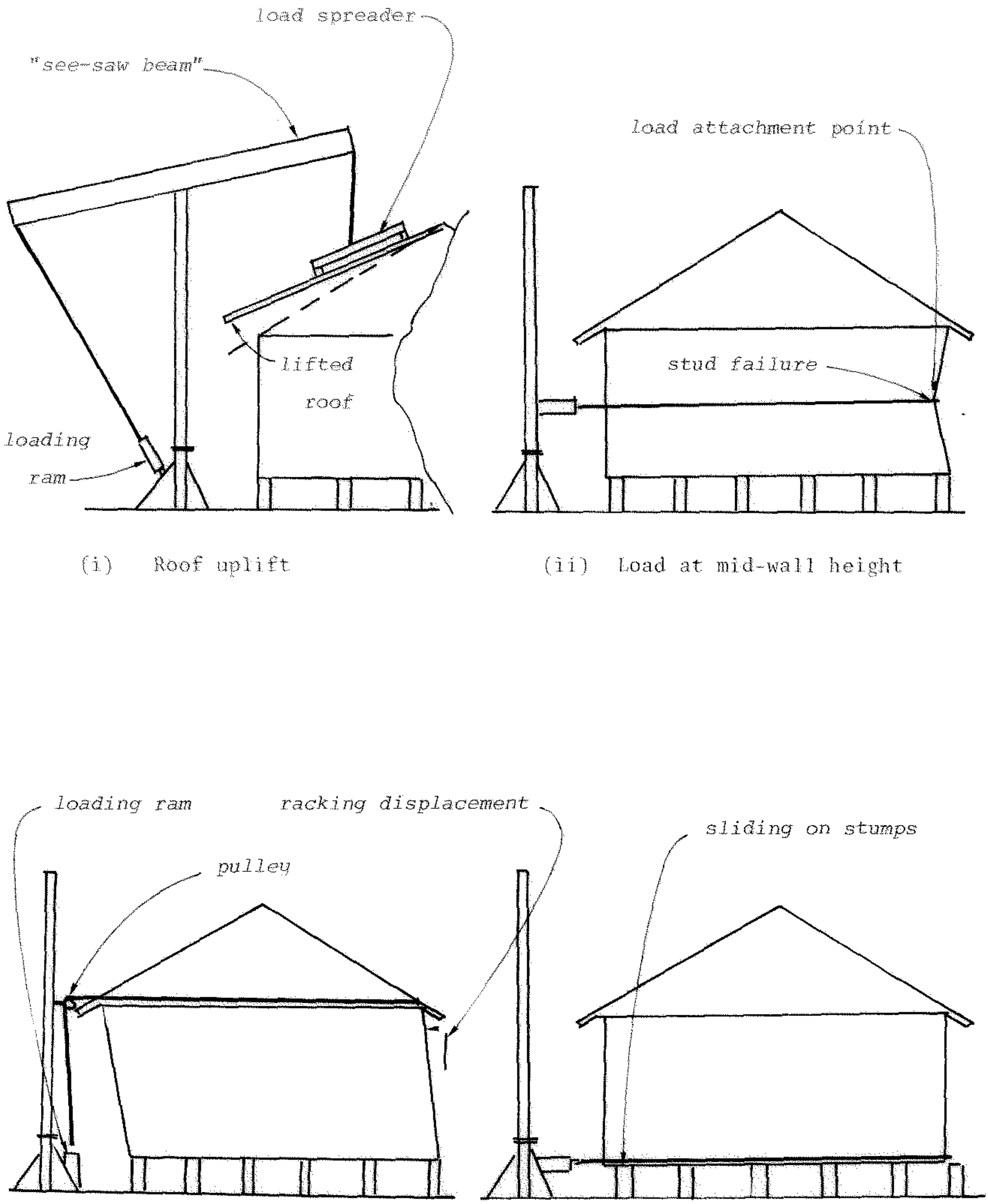


Figure 1. Plan and elevation of the building used in the first series of tests.

Roof sheeting	Corrugated galvanised steel sheeting, nailed every second corrugation.
Roof structure	Bolted timber trusses at approximately 3 metre centres with heavy timber purlins.
Ceiling	'Cane-ite' sheeting, nailed at 100 mm centres to a system of timber battens and ceiling joists.
External walls	Timber framed walls with weatherboards on outside and plywood on inside face.
Internal walls	Timber framed walls with plywood on all faces except those in the bathroom and laundry which were lined with asbestos-cement sheeting.

Loads were applied to the building using four steel reaction frames and hydraulic rams. The loads were applied to appropriate locations in the building using steel cables and the reaction frames in four different configurations. These are shown in Figure 2, and enabled the effects of uplift on the roof and lateral load on studs, roof structure and floor structure to be represented. Each test was conducted over a period of approximately one hour, and no attempt in this instance was made to use a repetitive loading program. The loads were measured using a load cell, and deflections using up to ten dial gauges. For lateral loads the orientation of the loading frames simulated pressure on the northern wall and suction on the southern wall of the house.

The tests are categorised as strength tests where a failure was specifically sought, and stiffness tests where deflections were limited to ensure elastic behaviour of the building. Each type of test will be treated separately in Sections 4 to 8. For all tests the casement windows and doors were left open so that no strengthening from these elements could be achieved.



(i) Roof uplift

(ii) Load at mid-wall height

(iii) Load at top of wall

(iv) Load at base of wall

Figure 2. Loading systems

3. EXPECTED WORKING LOADS - WIND TUNNEL TESTS

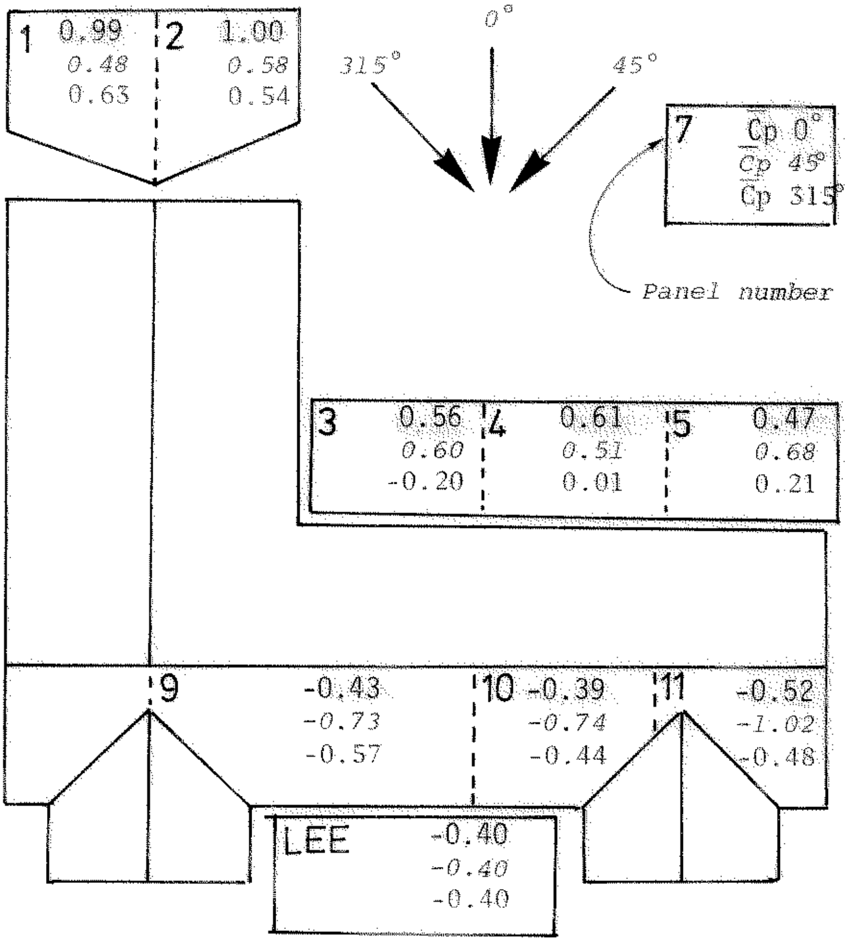
So that the performance of the house in the full-scale tests could be compared with performance expected in high winds, the actual wind loads for the building were calculated. A 1:50 scale model was constructed for testing in the James Cook University Boundary Layer Wind Tunnel, the design and operation of which has been detailed by Holmes (1977). The model was fitted with pressure taps on the northern walls and southern roof panels. These taps were manifolded together to enable the study of wind effects on panels approximately 4 m x 3 m in size on the prototype. Nine different wind directions were used:

(i)	wind from North - 0°	(vi)	wind at 60°
(ii)	wind from North East - 45°	(vii)	wind at 75°
(iii)	wind from North West - 315°	(viii)	wind at 330°
(iv)	wind at 15°	(ix)	wind at 345°
(v)	wind at 30°		

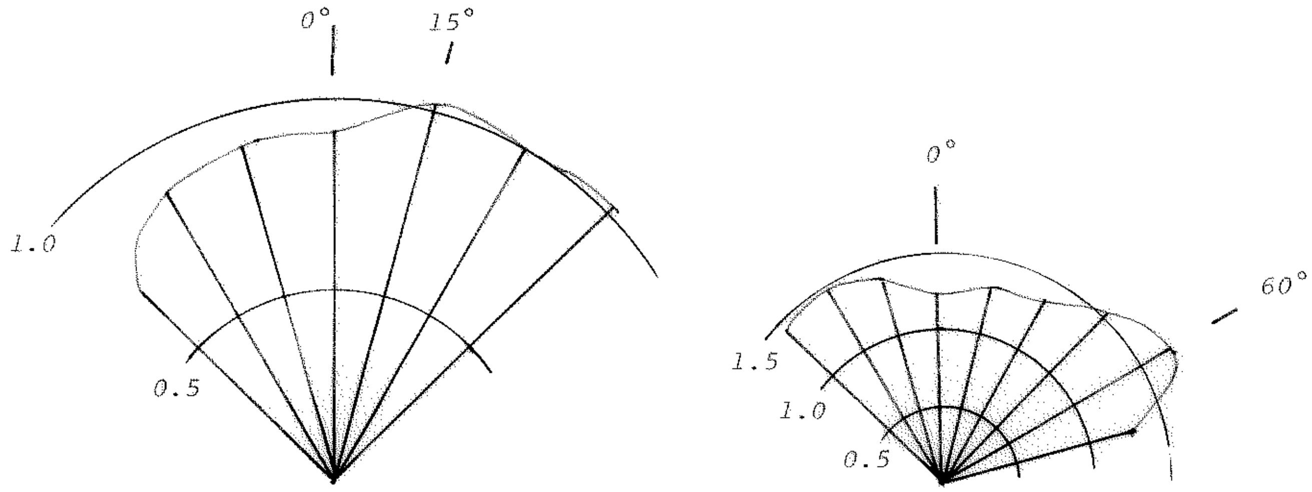
The average external pressure coefficients ($\overline{C}_p$) for the first three of these directions, and the leeward wall pressure coefficient from Best and Holmes (1978) are shown plotted on a plan of the house in Figure 3(i). The "L-shaped" plan of the house has a pronounced effect on the pressure of panels 3 and 4, with suction being experienced on panel 3 for winds from the North West (315°) and pressure for winds from the North East (45°).

To calculate total loads on the building, a design wind speed must be found and to obtain total roof uplift, an internal pressure must be assumed. The Australian Wind Loading Code (Australian Standards, 1981) establishes the terrain category 3, eaves height design wind speed for a 50 year return period in Townsville, to be 42 m/s. The internal pressure resulting in the highest uplift pressures on the roof is given as 0.8 (*ibid*).

The total northerly lateral load on the building; and the total uplift load on the roof panels 9 and 10 are shown plotted against wind direction in Figure 3(ii). The maximum total loads for a current code design wind event, designated 'AS code' can therefore be identified as:



(i) Mean pressure coefficients on 1:50 scale model



Total lateral load

Total uplift panels 9 and 10

(ii) Mean total pressure coefficients as a function of angle

Figure 3. Pressures on House

Load designation	Critical Wind Direction	Load (kN)	Net Average Pressure (kPa)
'AS code' lateral load on entire house (normal to Northern wall)	15°	57	1.04
'AS code' uplift on roof panels 9 and 10 (normal to roof)	60°	63	1.76

These 'AS code' loads can be compared with the maximum load experienced by the house in cyclone 'Althea'. This can be calculated from estimates of the maximum wind speed experienced during that cyclone. Trollope (1972) gives the peak gust velocity as 54 m/s (122 mph) as recorded by the Dynes anemometer at Garbutt airport (less than 1 km from the tested house). Using Table 4 of Australian Standards (1981) the equivalent eaves height velocity for terrain category 3 is 36 m/s. Trollope (1972) also indicates that for the most severe winds the wind direction varied from due North to due East. Thus the critical wind direction tabulated above correspond to the highest wind velocity directions experienced during cyclone 'Althea'. Cyclone 'Althea' loadings can be derived from the above velocity and the same pressure coefficients used to calculate 'AS code' loads.

Load Designation	Load (kN)	Average Pressure (kPa)	Fraction of Design Load
'Althea' lateral load on entire house (normal to northern wall)	42	0.76	0.73
'Althea' uplift on roof panels 9 and 10 (normal to roof)	46	1.29	0.73

The 'AS code' loads and estimated 'Althea' loads can be used as a comparison for loads obtained during the full scale tests.

4. UPLIFT ON ROOF - A STRENGTH TEST

As extensive testing of roof sheeting has been performed by this station, and other testing organisations and manufacturing companies no attempt has been made to apply uplift loads to the roof sheeting itself. Rather, the uplift loads were applied to the purlins of a 25 m^2 area of the roof using the loading configuration shown in Figure 2(i). The load spreaders distributed the applied uplift to twelve point loads on the purlins giving an average area distribution of just over 2 m^2 per load point.

The total ultimate load attained was 60.6 kN, equivalent to an average pressure over the test area of 2.4 kPa.

Test Result	'AS code'	'Althea'
Pressure (kPa) 2.4	1.76	1.29
Load factor = $\frac{\text{ultimate pressure}}{\text{'AS code' or 'Althea' pressure}}$	1.36	1.86

The load factors given above show the safety margin for the 'AS code' load and the 'Althea' load.

Throughout the test, the displacement of the purlins relative to the trusses was measured as indicated in Appendix A. At approximately 1.8 kPa uplift pressure, nails securing the purlins to the top chords of the trusses started to show evidence of pull out. The joint between the struts and the bottom chord of the truss showed some distress also at this load. As an inspection prior to testing had indicated that there was no evidence of this type of damage to the roof structure the maximum uplift pressure applied to the roof structure by cyclone 'Althea' must have been less than that value. This is in line with the 'Althea' loading derived in Section 3.

The load at which permanent distress to the structure occurred appeared to be close to the 'AS code' load, and the load factor relative to the 'AS code' load of 1.36 was lower than generally recommended load factors (Department of Construction, 1978). The tests indicate that performance in a current code design wind storm may not have been as satisfactory as the performance of the roof structure during cyclone 'Althea'.

The actual mode of failure was by withdrawal of the nails securing the purlins to the top chord nailing blocks. Forty nails in twenty locations were affected. This gave the average resistance of each nail at failure as 1.52 kN compared with the allowable design resistance of 0.63 kN (Australian Standard, 1975), i.e. average failure load was 2.4 times allowable load on that joint. The allowable load per joint gave a total permissible load on the test section of 1.01 kPa or 0.57 of the 'AS code' pressure. Thus it appears that the roof was significantly underdesigned with regard to current design rules. Appendix 4 to the Standard Building By Laws does not make a recommendation for holding down of purlins to trusses spaced at 3 metres, but holding down provisions for trusses at closer spacing are more substantial than those utilized in the tested roof.

The roof structure was significantly underdesigned, and although it performed satisfactorily in cyclone 'Althea's' loading, the tests have shown that deterioration in an 'AS code' cyclone would be expected. As significant nail pull out occurred at the design load, the action of cyclic loading as experienced in high wind events, may well have decreased the load factors obtained in the test.

Failure of the roof structure was sudden and simultaneous release of the twenty loaded joints. A failure of this kind during high wind conditions would have resulted in the sudden removal of a large part of the roof sheeting with purlins attached which would have become a very dangerous single piece of airborne debris.

5. LATERAL LOAD AT MID WALL HEIGHT - STRENGTH TESTS

Wind pressures on the windward wall act to bend the wall inwards. The pressures applied to the external cladding and suctions on the internal cladding transfer load to the wall studs by bending of the cladding material. Then lateral loads on the studs cause them to bend inwards transferring load to other elements in the wall. These may include door and window lintels and jambs or wall noggings, but certainly will include top and bottom wall plates.

A series of tests on wall studs was performed to ascertain the degree of load sharing carried out between studs and other elements in walls, and to determine the factors that affect the in-situ strength of studs. In these tests, the studs were loaded at mid height using the loading configuration as shown in Figure 2(ii). Three tests were performed with internal and external claddings in place, as detailed in Sections A.3 to A.5, and two tests were performed on studs after all cladding, timber bracing and noggings had been removed, as detailed in Section A.6 and A.7. As there was some doubt as to the validity of the load readings for the latter two tests, five timber samples taken from studs in the house were subjected to flexure tests to determine the modulus of rupture and the modulus of elasticity of the stud timber. The results of these tests are tabulated below and can be compared with stud tests 4 and 5.

	Test No.	Modulus of Rupture (MPa)	Modulus of Elasticity (MPa)
Sample Tests	1S1	89	8587
	1S2	78	8466
	2S1	64	10236
	2S2	75	9229
	3S1	80	9981
	Average	77	9300
Stud Tests	Stud 4	32	4980
	Stud 5	98	5980

Table of Material Properties of Stud Timber

The results of the sample tests indicate that the timber used in the studs was most probably F8 hardwood. The modulus of rupture obtained in stud test 5 also supports this estimate, but the two stud test values of modulus of elasticity and the modulus of rupture shown in stud test 4 are approximately half the expected values. As noted in Sections A.6 and A.7, the load values recorded were suspect for all of stud test 4 and the first part of stud test 5. Therefore, for the remainder of this section it will be assumed that the studs were of F8 hardwood.

For the first three stud tests, in which the wall claddings, nogging and bracing were left intact, a theoretical finite element analysis of the timber elements in the walls was performed. This showed that most of the load sharing was due to bending of the cladding. In this case the external cladding consisted of substantial timber weatherboards, so significant load sharing was achieved in the tests on external walls. However some conclusions on the load transfer between studs can be drawn.

(i) The bulk of the load transfer was by bending of the cladding with minor contribution by membrane action of the cladding and by bending of noggings and bracing. At the present time it is possible to quantify the distribution, but as the external cladding used is not typical of modern claddings, a presentation of the analysis of the tested house would not be relevant to current building practice.

(ii) The timber framework around openings is capable of attracting and distributing lateral load across the openings to other studs.

(iii) The amount of lateral load transfer between studs is very much dependent on the flexural properties of the cladding with the weatherboard walls proving much more capable in this respect than the internal wall with two plywood clad faces.

(iv) Walls at right angles to the loaded walls effectively pin the loaded wall at that point and attract load from the cladding.

In Figure 4, a plot of the deflected shape of the wall as predicted by the finite element program is shown. The transfer of load horizontally by bending of the external cladding can clearly be seen as well as the bending of studs to transfer load to top and bottom plates. The effect of a transverse wall in reducing the deflection at its junction with the loaded wall can also be seen.

In loading the walls at midheight, it was recognised that whilst this is highly likely for external walls, the combination of broken windward and leeward windows, and closed internal doors is required to achieve a wind loading on internal walls. Even so, a few isolated cases of failure of internal wall details were observed in Darwin after cyclone Tracy. The wind loading on internal walls appears a valid load case. An internal wall stud was therefore tested to failure along with two external wall studs.

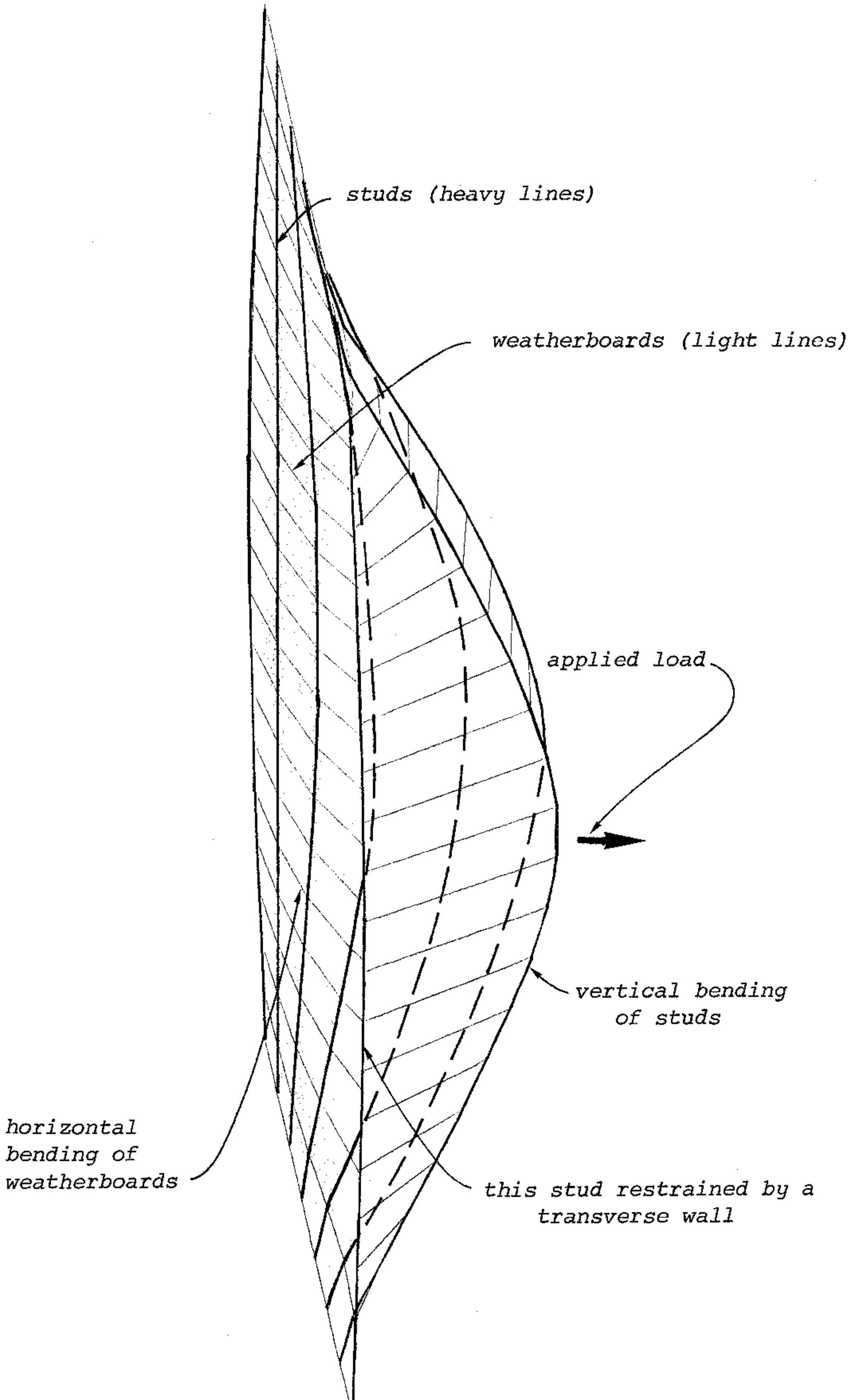


Figure 4. Deflected shape of wall during stud tests

The bending moments applied to the stud at failure are shown below and load factors have been calculated to compare these failure loads with those expected in the 'AS code' return period wind and in cyclone 'Althea'. An internal suction equal to the suction on the leeward wall has been assumed in the calculation.

Stud Test	Wall	Cladding	Bending Moment at Failure kNm	Load Factor 'Althea'	Load Factor 'AS code'
1	external	weatherboards/ plywood	14.7	46	32
2	external	weatherboards/ plywood	9.1	28	20
3	internal	plywood/ plywood	9.8	30	21
4	external	none	0.7	2.1	1.5
5	external	none	4.8	15.0	10.5

None of the studs tested could be described as perfect. Many displayed significant cross grain and failure was precipitated at knots in tests 1 and 3 and at notches in test 4. Even so the load factors with respect to both cyclone 'Althea' and 'AS code' return period loads were high, commensurate with the observed performance of studs subjected to high wind loads. In many cases where stud failure details have been observed in high winds, the stud timber has remained intact and fastening with bottom or top plate has proved insufficient to carry the load. However, all of the studs tested failed by timber flexure failure near the point of maximum moment, and the end driven nails at both top and bottom plate connections bent, but still carried the required loads.

6. LATERAL LOAD AT TOP PLATE LEVEL - STRENGTH TESTS

As discussed in Section 5 of this report, the lateral loads on the walls of the house are transferred by bending of the studs to the top and bottom wall plates. Those loads that are carried to the top wall plate must be taken to ground by another mechanism - usually the bracing resistance of walls aligned nearly parallel to the direction of the lateral force. A rather complex force transfer mechanism at or above the top plate level distributes load from the top of the studs to the bracing walls. This mechanism receives attention in Section 8, and in this section its load carrying capacity will be discussed.

In a series of tests designated Top Plate Tests 1 to 5 and summarised in Sections A.10 and A.11, lateral point loads were placed on the top plate of the Northern wall of the building using a loading system as shown in Figure 2(ii). In each of Top Plate Tests 1 to 4 a point load of approximately 3 times the 'AS code' lateral load over the same area was applied to the house at 4 separate locations with some deviation from linear behaviour but no sign of failure. However, in each test, the nails securing the ceiling sheeting to the roof structure showed evidence of movement in the cane-ite sheeting, especially immediately adjacent to the loading points. An inspection of the roof structure also showed that trusses near the loading point had moved, causing significant displacement of the purlins next to those trusses that had moved. No tearing of the roof sheeting was evident, even though the deflected shape of the purlins indicated that shear was being transferred to the sheeting.

In Top Plate Test 5, load was simultaneously applied at the same loading points used in Top Plate Tests 1 to 4. In this case, the deflections at the points monitored were the sum of deflections obtained in the previous four tests, and the same slight deviation from elastic behaviour was observed. However in this case a failure of a top plate and an area of approximately 2 m^2 of ceiling occurred in the vicinity of one loading point. The top plate failed in flexure, 300 mm from one load point at a notch and it was postulated that the ceiling failed as it could not carry the additional load imparted when the top plate failed. As the load at the failure location was applied as a point load more than 1 m away from the nearest internal wall, the bending moment in the top plate induced during the test would have been higher than that induced during a high wind event where the load on the top plate would be approximately uniformly distributed. The load factors shown below are therefore probably slightly conservative, as a uniformly distributed loading would have resulted in higher failure loads.

	Test Result	'AS code'	'Althea'
Total lateral load on Northern wall (kN)	192	57	42
Load factor = $\frac{\text{ultimate load}}{\text{'AS code' or 'Althea' total load}}$		3.37	4.57

As the test results represented a lower bound as indicated above, the load factors quoted above also represented a lower bound. Even so the load factor obtained were well in excess of those generally regarded as acceptable (Department of Construction, 1978), indicating that the roof structure has ample capacity to carry lateral wind loads to bracing elements. In carrying lateral load away from the load points, the deflected shape of the purlins and ceiling joists indicates that both the roof sheeting and ceiling material act as diaphragms.

Even at loads in excess of three times the 50 year return period lateral wind load, there was no sign of permanent damage to the roof sheeting and damage to the ceiling was restricted to local effects around the point loads used in the test. The roof structure therefore demonstrated its effectiveness in carrying lateral loads to bracing walls. Walker (1975), Reardon and Oliver (1982) and Beck and Morgan (1975) all make the point that after the roof structure had been removed by high winds, the walls were often not capable of resisting the lateral load imparted by the wind.

Recognising the importance of the bracing elements in the roof in transferring lateral loads to bracing walls, Appendix 4 of the Queensland Home Building Code gives construction details for the fixing of the top of bracing walls to the roof structure so that lateral force transfer is enabled, yet allowing for limited vertical movement of the roof structure. Also a program of continuing research into the strength of both ceiling diaphragms (Walker and Gonano, 1981) and roof sheeting diaphragms (Nash and Boughton, 1981) will assist in predicting the strength of the roof structure bracing. The indications of this test are that there is much potential for utilizing roofing and ceiling cladding elements to transfer lateral loads to load carrying members within the building.

7. LATERAL LOAD AT FLOOR LEVEL - A STRENGTH TEST

As indicated in Sections 5 and 6 of this report, all lateral loads on the building are eventually transferred to floor level either by studs directly to the bottom plate or by studs to roof structure bracing systems and then to the floor through walls parallel to the direction of the wind. Thus, all lateral load applied to the building must be transferred from the floor to the ground. In the case of the house reported in this work, this involved the transfer of force from the timber floor bearers through steel ant caps to timber piles, and then to the ground. As there was no effective shear connection between the house and the stumps, the transfer of this lateral force relied on friction between the bearers and the ant caps and between the ant caps and the stumps.

In this test, an 11.5 metre length of house with weight estimated at 140 kN was subjected to a lateral load applied at floor level as shown in Figure 2(ii). At locations not in line with the two load points, the floor showed very little movement until the total lateral load was near 60 kN. Significant displacement of the test portion of the house was experienced in the direction of the load until the ultimate load of 85 kN was achieved. The coefficient of friction at first slip was 0.43 and at ultimate load 0.61. The generally accepted value of the coefficient of friction for cast iron on hardwood, 0.49 (Oberg and Jones, 1954) lies midway between the upper and lower bounds for the coefficient of friction established in the full-scale test. At the ultimate load obtained in the test, most of the movement occurred between either the timber bearers and the steel ant caps or between the steel ant caps and the timber piles. In two cases, the movement was accommodated by rotation of the pile in the ground. Thus the main part of the resistance to the lateral load was provided by friction between steel and timber.

The load obtained in the test can be compared with the expected working load for current code design winds and the 'Althea' load given in Section 3, however in order to make the comparisons valid for high wind conditions it is necessary to make allowance for the uplift on the roof, reducing the normal force at stump level during high winds, hence reducing the available lateral force resistance.

As the lateral loads used in determining the current code design load and the 'Althea' load were for wind at 15°, the uplift used must also correspond to wind at 15°. At this angle, the mean uplift pressure coefficient on the southern side of the roof was 0.47 and the mean pressure coefficient on the northern side of the roof 0.1 downward (Holmes, 1981). This gives a net uplift on the 11.5 m section of the house tested of 18 kN under 'AS code' conditions, and 13 kN under cyclone 'Althea' loading. The dead weight of the structure can be reduced by the amount of net uplift to give the total normal load on the stumps, and assuming that coefficient of friction is independent of normal loading, the lateral load at first slip and at failure can be found for each of the 'AS code' conditions and during cyclone 'Althea'.

First Slip Loading				
	'AS code'		'Althea'	
	Derated Test Result	'AS code'	Derated Test Result	'Althea'
Lateral pressure	1.63	1.04	1.71	0.73
Load factor	1.57		2.34	

Ultimate Loading				
	'AS code'		'Althea'	
	Derated Test Result	'AS code'	Derated Test Result	'Althea'
Lateral pressure	2.31	1.04	2.42	0.73
Load factor	2.22		3.32	

The load factors calculated above indicate that the capacity to resist lateral load during cyclone 'Althea' without slipping was well over two times the actual lateral load applied. Indeed from the paint marks on the ant caps around the external wall it appears that no sliding of the house on the piles had taken place in the life of the building.

However the current code design load factor for first slippage is lower than recommended (Department of Construction). Thus under the cyclic loading applied by a current code design high wind event, some slippage may have commenced and if the loading had persisted, a failure may have eventuated.

Timber framed houses that are built to modern specifications tend to have lighter frames and ceiling structures. Also, there are many cladding systems available for external walls that are considerably lighter than the weatherboards on the tested house. The trend towards shallower pitched roofs generally increases total uplift on the structure and the lighter construction material used also serves to reduce the normal load at the base of the structure. Therefore, friction cannot be relied upon to ensure the lateral stability of a house. Appendix 4 to the Standard Building By-Laws (Qld) specifies minimum bracing requirements below floor level and details lateral load transfer mechanisms from the floor level to the top of bracing elements. It is to be noted that cranked anchor rods bolted to the sides of the timber piles as used on the tested house, totally without any effect on the lateral load resistance of the structure, have been excluded as lateral load transfer devices in Appendix 4.

It is clear that the dead weight of a house is not necessarily sufficient for friction to be relied upon to resist the lateral forces applied to houses in high wind events. This has also been shown in investigations of damage to housing following Cyclone 'Tracy' Walker (1975), Cyclone 'Isaac' Reardon and Oliver (1982), and many others. The attention to lateral load transfer details below floor level as indicated in Appendix 4 is very necessary.

8. LATERAL LOAD STIFFNESS TESTS

A series of six site tests and two laboratory tests was designed specifically to determine the force path through the structure for loads applied at top plate level. As discussed in previous sections, these loads must be transmitted to ground by a number of different bracing elements, including roof sheeting, ceiling material and wall claddings. For the site tests, lateral load was applied at a single point to the Northern wall at top plate level as shown in Figure 2(ii). Displacements were monitored at a number of points along the Northern wall and loading was restricted to keep the house behaviour within elastic limits. Load was applied at the same point a number of times, and after each test a potential load carrying element was removed. By analysing changes in the stiffness of the building, the contribution of each element removed to the load carrying mechanism of the whole house could be determined. The pattern of element removal was as follows.

Test	Sections applicable	New element removed prior to test
Top plate Test 2	6, A.10	House intact
Top plate Test 6	4, 6, A.10, A.12	Removal of some roof sheeting and purlins on Southern side of roof in roof test (Sec. 4). Minor damage to ceiling 3.1 m from load point (Sec. 6).
Top plate Test 7	8, A.12	Removal of weatherboards from Northern wall
Top plate Test 8	8, A.12	Removal of all ceiling sheeting
Top plate Test 9	8, A.12	Removal of all roof sheeting from Southern side of the roof
Top plate Test 10	8, A.12	Removal of roof sheeting from Northern side of the roof
Top plate Test 11	8, A.12	Removal of all ceiling battens

The two laboratory tests were performed on elements removed intact from the building. The loads applied to the house in the tests above were at top plate level, immediately beside an internal wall that was parallel to the applied load. This wall was removed after the site testing was completed, and tested in the laboratory as detailed in Section A.12. The loading system was designed to approximate the site test loads as closely as possible. In the second laboratory test, a 3 m x 3 m area of ceiling complete with battens and joists was removed intact and loaded as shown in Section A.12. This enabled the stiffness of the ceiling diaphragm to be evaluated.

8.1 Lateral Load Resistance of 'Bracing Walls'

In examining the construction of the house, it was evident that no walls had received special treatment as 'bracing walls'. All internal and external walls were fixed to the ceiling system and the roof trusses by nails through ceiling joists and ceiling battens to the top plates. All wall claddings were fastened to wall frames by nailing at close centres. Thus all internal and external walls that were parallel to the direction of the applied load were considered potential bracing walls.

To evaluate the stiffness of these walls in resisting lateral loads, two methods were used.

(i) Laboratory test.

The laboratory test on the wall removed from the house gave a stiffness that varied with the support condition for the wall. In the house, the wall was nailed directly to the timber floor which was not a very stiff support. The laboratory stiffness of the wall was evaluated as 0.33 kN/mm deflection at the load point per m length of wall, but due to differing support conditions the in-house stiffness was expected to be less than this value.

(ii) Evaluation of site test results.

By assuming that all walls parallel to the load were bracing walls and that load resistance was proportional to the deflection of the top of the wall and also to the unbroken length of the wall, the performance of the house in all the top plate tests could be used to evaluate the wall stiffness. By equating forces and moments in each of these 11 tests, the calculated wall stiffness ranged from 0.20 to 0.24 kN/mm top deflection per m length of wall. The mean was 0.22 kN/mm per m. This result was considered likely in view of the comments on the laboratory test support stiffness made above.

The lack of scatter in the derived results for wall stiffness which come from many different loading and reaction configurations indicates that the assumption of a common wall stiffness proportional to the unbroken length of the wall was largely valid for the whole house. The value of 0.22 kN/mm per m was used for the succeeding analyses.

8.2 Lateral Load Distribution in Roof Structure

The point load applied to the house in the stiffness tests was located adjacent to an internal wall. From the above analyses, it was determined that in the intact house 30% of the applied load was carried to the floor level by that wall. This left 70% of the applied load to be distributed to other walls by elements in the roof structure. The load transferred to each bracing element at the load point and the load transferred to each bracing wall was determined by using differences in the deflections at each wall for top plate tests 6 to 11. This yielded the following load distribution.

Item	% of total lateral load carried by element in intact house	% of roof structure lateral load in intact house
Loaded internal wall	30	-
Weatherboards	3	-
Roofing sheeting system	33	49
Ceiling system	29	43
Bending of walls perpendicular to applied load	5	8

A diagram showing the load transfer between the top plate and each item listed above is shown in Figure 5.

It is clear from this diagram that the bending action of the weatherboards and the bending action of internal walls only spanned very small distances - at most the distance to the nearest bracing wall on each side. As the mechanism involved in the action of both of these elements was bending, the effectiveness of the elements decreased dramatically with an increase in the distance between bracing walls. The action of these two elements in resisting lateral forces is therefore only of any significance in very small rooms such as toilets or laundries, and in these rooms the ceiling diaphragm is also very stiff, and also attracts much lateral load. As weatherboards of the type used on the house in 1942 are not commonly used in modern construction, and the bending of internal walls is dependent on the roof trusses being fixed to all internal walls, a practice generally precluded

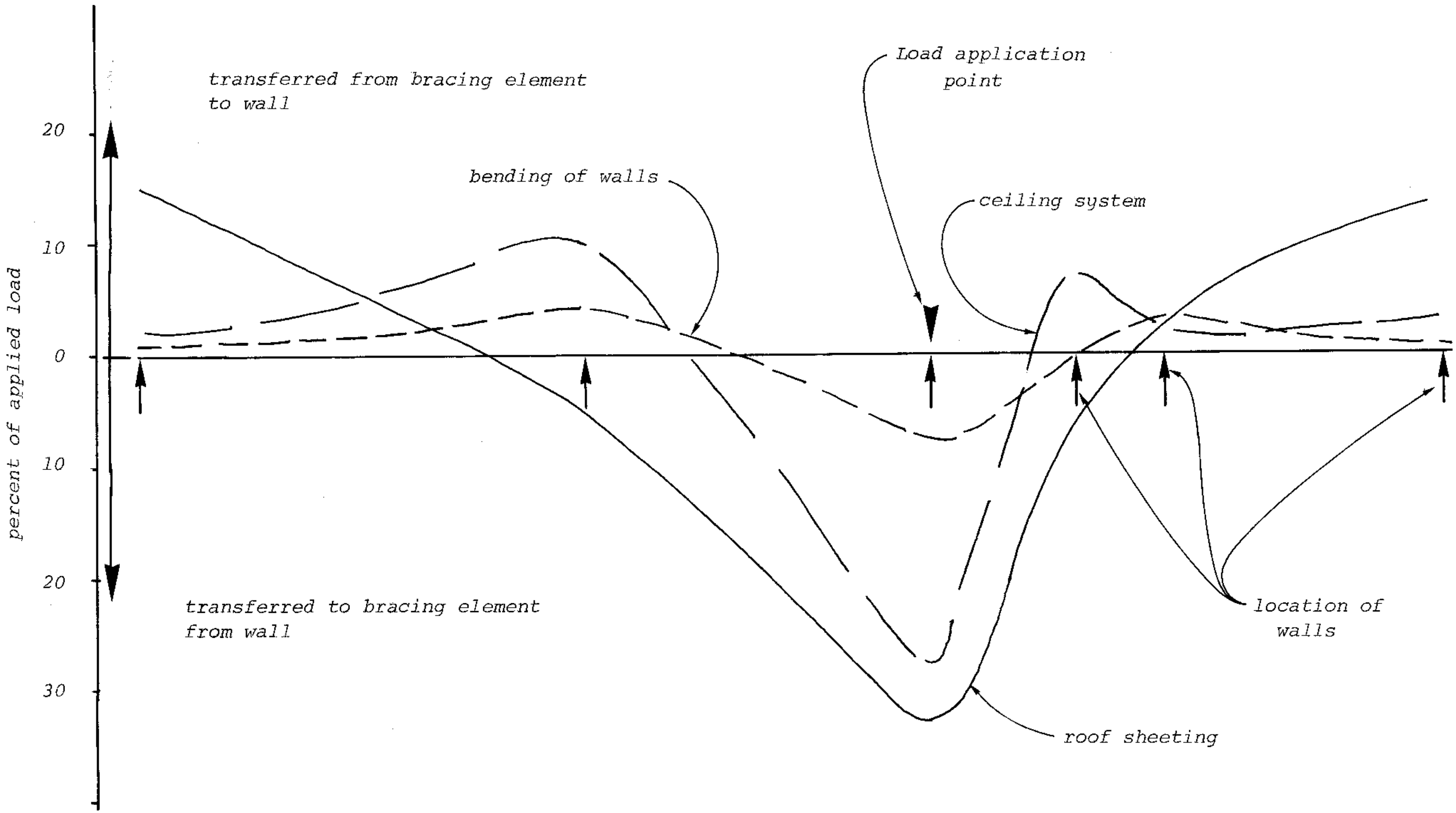


Figure 5. Percentage of applied load carried by lateral bracing elements.

in most modern building by-laws, it is recommended that the bending action of internal walls and external cladding not be relied upon for any bracing strength.

This leaves only two bracing elements to distribute lateral loads on the house to the bracing walls, the ceiling diaphragm and roof sheeting bracing.

The ceiling diaphragm also transmitted load over relatively short distances in this case as can be seen in Figure 5. The largest part of the load carried from the load point by the ceiling is transferred to the closest walls to the load point. However, in this case the load transfer mechanism is one of direct shear, and the load carrying capacity of the diaphragm does not decrease as dramatically with span when compared with a bending mechanism. The stiffness of the ceiling diaphragm as evaluated in the laboratory test was 0.12 kN/mm and from the house test results adjusted to conform with the same geometry as the laboratory test, 0.11 kN/mm. This indicates that in-situ the ceiling diaphragm as loaded in the house tests did carry load primarily in shear. In modern construction it is still common practice to apply ceilings to each room individually. The diaphragm action of the ceiling then would satisfactorily transfer load to adjacent walls and as the stiffness of recently tested ceiling diaphragms (Walker, Boughton and Gonano, 1982) was greater than that used in the tested house the percentage of load attracted to the ceiling would be greater than that carried by the tested ceiling. However the mechanism of load transfer may differ from the pure shear transfer postulated here under different loading conditions. This action will be further persued in laboratory and theoretical studies, and also in future house testing.

The roof sheeting appeared capable of transmitting loads over quite large distances, being loaded not only by the wall to which the load was applied, but also by the walls to which the ceiling diaphragm carried load. This made the analysis of the mechanism of the roof sheeting bracing more difficult and in fact the inaccuracies associated with creep during measurement swamped efforts to calculate the diaphragm's stiffness. Only qualitative conclusions about the roof sheeting bracing action can be drawn. The roof sheeting system forms a stiff bracing element capable of transmitting lateral loads the length of the unbroken roof line, by a combination of batten bending and roof sheeting shear. Further theoretical and laboratory work is being performed on the roof diaphragm to ascertain the mechanism used in load transfer, and it is planned to check this work in the next house tested.

The high percentage of load carried by the roof sheeting, and the large distances it can span make it a very useful lateral load carrying diaphragm.

9. RECOMMENDATIONS FOR FUTURE TESTING

The analysis of the test results has highlighted the need for improving testing procedures in future full scale house tests. Most of these stem from inadequacies in the measurement system used in the first full scale tests.

In order to perform the analysis summarised in Section 8 of the publication, some interpolation of results was required as there were only ten dial gauges available for use on the project and deflections at 8 points on the top plate and 8 points on the roof structure were required for the analysis. The use of more deflection measurement points will eliminate interpolation and help to reduce the error build up in the analysis.

The use of electronic remote-reading deflection measurement equipment will ensure that readings are obtained with little creep, and no movement due to the efforts of the gauge readers in reading the gauges.

The use of thirty to forty electronic remote-reading deflection measurement devices will minimise error accumulation in performing the analyses and enable a more accurate determination of force paths within the building.

Similarly in performing stiffness testing as detailed in Section 8 the use of more than one load application point would enable the checking of mechanisms under different loading conditions.

Throughout the testing programme it is desirable to ensure that loading conditions are as realistic as possible. It is widely recognised that high winds impart fluctuating loads on buildings, and in performing strength tests it is therefore important to include a cyclic load testing sequence. This will highlight any tendency of components to work loose or suffer from fatigue. Ultimately it is hoped that it will be possible to apply cyclic loads to a whole house.

10. CONCLUSIONS

This series of tests has demonstrated the viability of the house testing project in that higher loads than those expected in the current code design tropical cyclone in Townsville have been applied to the structure. Weaknesses within the structure were identified, and the analysis of a series of stiffness tests has enabled a qualitative statement on force paths in the building to be made.

Conclusions can be drawn on the performance of the house tested and also on the future of the house testing project.

10.1 Conclusions on the Performance of the Building

Although the house had withstood three separate tropical cyclones in its 40 year history, the tests uncovered some structural deficiencies which may have affected its performance in a current code design wind event.

The purlin to top chord of truss connection showed signs of distress at the current code design wind load. This distress may have been aggravated by cyclic loading and could have led to the demise of the roof. The total load on the roof at the time of failure of the purlin to top chord connection gave a load factor over the current code design considerably less than that deemed acceptable. The detail that gave rise to the failure fell considerably short of current building by-law requirements.

The reliance of building weight to prevent sliding on the stumps also gave an unsatisfactory load factor over the current code design wind. The tendency to slide may have been aggravated by the fluctuating loading experienced in extreme high wind conditions and led to failure. The detail employed in the building at the stump bearer junction would have proved unacceptable using modern building by-laws.

In contrast, the strength of the studs in bending appeared to be well in excess of that required to resist current code design winds. The intact roof structure also had ample strength to distribute lateral forces through the building to bracing walls. Both of these findings are in agreement with observations made in damage reports on extreme wind events.

10.2 Conclusions on the House Testing Project

The first series of tests has established the success of a portable house testing method to simulate high winds.

The results have highlighted the need for research into ceiling and roof sheeting diaphragm, and future testing will continue to steer research energies within the Australian building industry. The provisions of current building by-laws were compared with building details in the house tested, and the extra attention given to purlin to truss connections and bearer to stumps connections as given in current building codes justified. Future tests on houses constructed to comply with current codes will enable the checking of adequacy of code requirements and evaluation of proposed changes to the codes.

ACKNOWLEDGEMENTS

Without the generous donation by the Queensland Housing Commission of the house used in this series of tests, the house testing project would not have been made possible. The author is grateful to Mr. M.J. Hutchins, the Commissioner at that time for his help.

The work of Mr. W. Larcombe in developing computer programs used in the analysis of the stud tests is also gratefully acknowledged.

REFERENCES

1. AUSTRALIAN STANDARD (1975) - 1720, SAA Timber Engineering Code, Standards Association of Australia.

2. AUSTRALIAN STANDARD (1981) - 1170 Part 2, SAA Loading Code, Part 2 - Wind Forces, Standards Association of Australia.

3. BECK, V.R. and MORGAN, J.W. (1975) "Appraisal of Metal Roofing under Repeated Wind Loading - Cyclone Tracy, Darwin 1974". Housing Research Branch Technical Report No. 1, Australian Department of Housing and Construction, February 1975.

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

5. BOUGHTON, G.N. and REARDON, G.F. (1982) "Simulated Wind Tests on a House - Part 1 Description". Technical Report 12, Cyclone Structural Testing Station, James Cook University of North Queensland.

6. DEPARTMENT OF CONSTRUCTION (1978) "Guidelines for the Testing and Evaluation of Products for Cyclone-Prone Areas". TR 440, Experimental Building Station, Sydney.

7. HOLMES, J.D. (1977) "Design and Performance of a Wind Tunnel for Modelling the Atmospheric Boundary Layer in Strong Winds". Wind Engineering Report 2/77, James Cook University of North Queensland.

8. HOLMES, J.D. (1981) "Wind Pressures on Houses with High Pitched Roofs". Wind Engineering Report 4/81, James Cook University of North Queensland.

9. NASH, L.M. and BOUGHTON, G.N. (1981) "Bracing Strength of Corrugated Steel Roofing". Technical Report No. 8, Cyclone Structural Testing Station, James Cook University of North Queensland.

10. OBERG, E. and JONES, F.D. (1954) "Machinery's Handbook". 15th Edition, The Industrial Press, New York, p. 517.

11. REARDON, G.F. and OLIVER, J. (1982) "Report on Damage Caused by Cyclone Isaac in Tonga". Technical Report No. 13, Cyclone Structural Testing Station, James Cook University of North Queensland.

12. TROLLOPE, D.H. (1972) "Cyclone 'Althea', Part 1 - Buildings". James Cook University of North Queensland.

13. WALKER, G.R. (1975) "Report on Cyclone 'Tracy'", James Cook University of North Queensland.

14. WALKER, G.R. and GONANO, D. (1981) "Investigation of Diaphragm Action of Ceilings - Progress Report 1". Technical Report No. 10, Cyclone Structural Testing Station, James Cook University of North Queensland.

15. WALKER, G.R., BOUGHTON, G.N. and GONANO, D. (1982) "Investigation of Diaphragm Action of Ceilings - Progress Report 2". Technical Report No. 15, Cyclone Structural Testing Station, James Cook University of North Queensland.

16. DEPARTMENT OF LOCAL GOVERNMENT QUEENSLAND. (1982) "Appendix 4 to the Standard Building By-Laws 1975". Home Building Code Queensland.

APPENDIX A - TEST RESULTS

A.1 Loading Arrangements

Figure A.1 shows a plan of the house showing the position of the loading frames and giving the location of the points of application of the loads.

Figure A.2 shows an elevation of the house giving more detail on the position of the load application points.

Loads were applied using hydraulic tension rams which applied loads to the house via a system of steel cables and a yoke. The loads were applied to frame members through the cladding material. In most of the tests, the load was measured using a compression load cell placed between the yoke and the cladding as shown in Figure A.3(i). A steel spreader plate was placed against the cladding to prevent undue local damage to the cladding caused by high bearing stresses at the load cell. In some of the tests where multiple loadings were used, the hydraulic pressure in the rams was measured and this could be related to the applied load using a laboratory calibration curve for each ram and gauge.

A.2 Deflection Measurement

Deflections were measured using dial gauges mounted on rigid scaffolding erected separate from the building on the ground. The dial gauges were fixed as shown in A.3(ii) so that a rapid or unexpected failure would give rise to movement away from the gauge. This in most cases protected the gauges from damage.

The recorded deflections were stored on a computer on site, and load-deflection curves produced while the tests were in progress. In this way, non-destructive tests could be restricted to the elastic behaviour region, and warning of failure could be given in tests to destruction.

Points A to E	*stud loads*
F to I	*top plate loads*
J and K	*bottom plate loads*
L and M	*uplift loads*

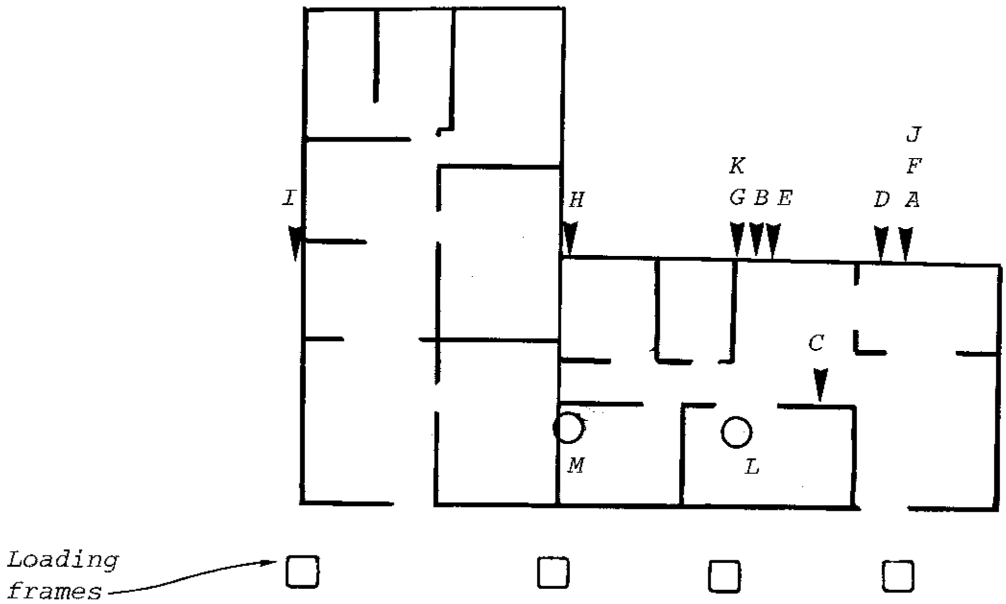


Figure A.1 Plan of house showing load points

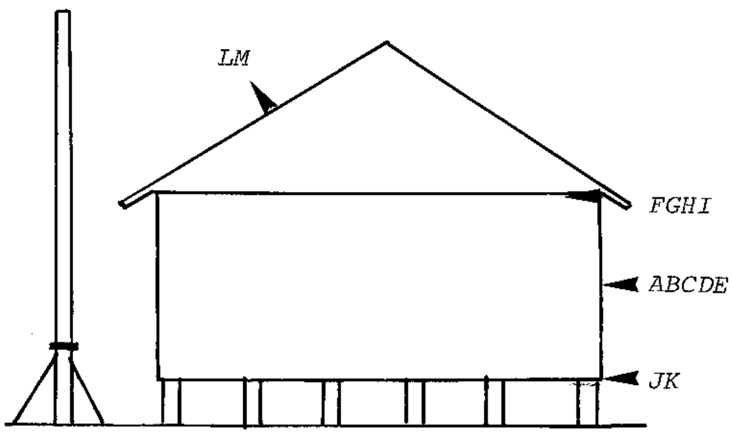
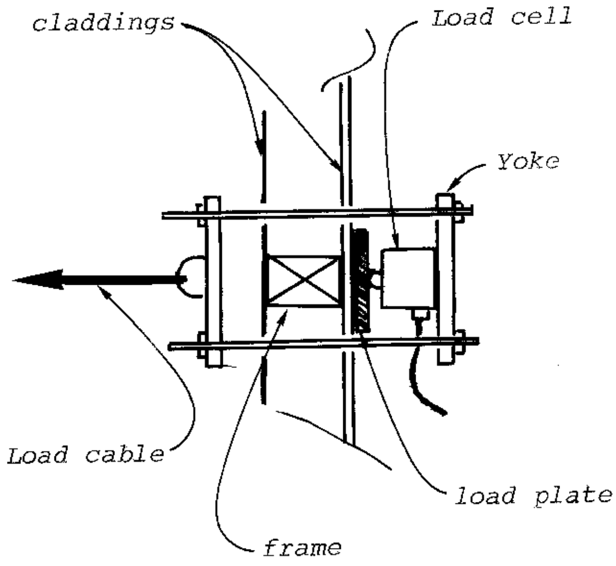
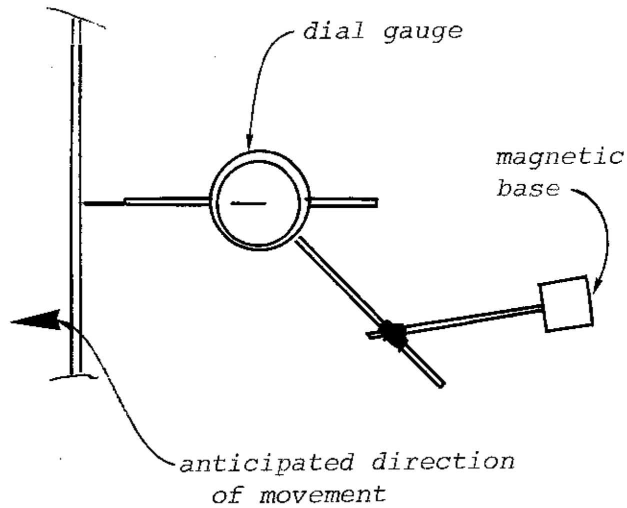


Figure A.2 Elevation of house showing load points



(i) load measurement



(ii) deflection measurement

Figure A.3 Monitoring loads and deflections

A.3 Stud Test 1

Geometry of Test

In this test a stud in an external wall was loaded at mid height with the load applied at point A as shown in Figures A.1 and A.2. The external cladding and internal cladding was left intact, with small holes drilled either side of the loaded stud to permit the installation of the yoke as shown in Figure A.3(i).

Deflection gauges were positioned as shown in Figure A.4. The positions shown enabled the deflected shape of the stud to be found and also allowed the evaluation of the sideways distribution of load by the cladding and the noggings.

Test History

The load was applied in 1 kN increments and deflections measured at all gauges for each increment up to 14 kN, then the gauges were removed for their protection as failure appeared imminent. Loading continued until failure.

At low loads, the weatherboards were drawn inwards as the stud deflected. The studs adjacent to the loaded stud also deflected inwards, but there was little deflection at the position of the internal wall. There was significant deflection in the direction of the applied load on each side of the window.

At an applied load of 6 kN, the nails securing the weatherboards to the studs commenced to pull out of the loaded stud in the vicinity of the load. The pull out was gradual and progressed slowly away from the load point up and down the stud.

The ultimate load was reached at 21 kN when the loaded stud failed in flexure 200 mm below the load point at a knot in the hardwood. As a post failure effect the stud deflected excessively, breaking the plywood internal lining and pulling away from the weatherboards over its entire length. The nails securing the stud to the top and bottom plates remained intact throughout the test.

A load-deflection curve for the load point is shown in Figure A.5.

A.4 Stud Test 2

Geometry of Test

This test was very similar to Stud Test 1. The external wall stud was loaded at mid height at point B on Figure A.1. Deflection gauges were placed in similar positions to those utilized in Stud Test 1.

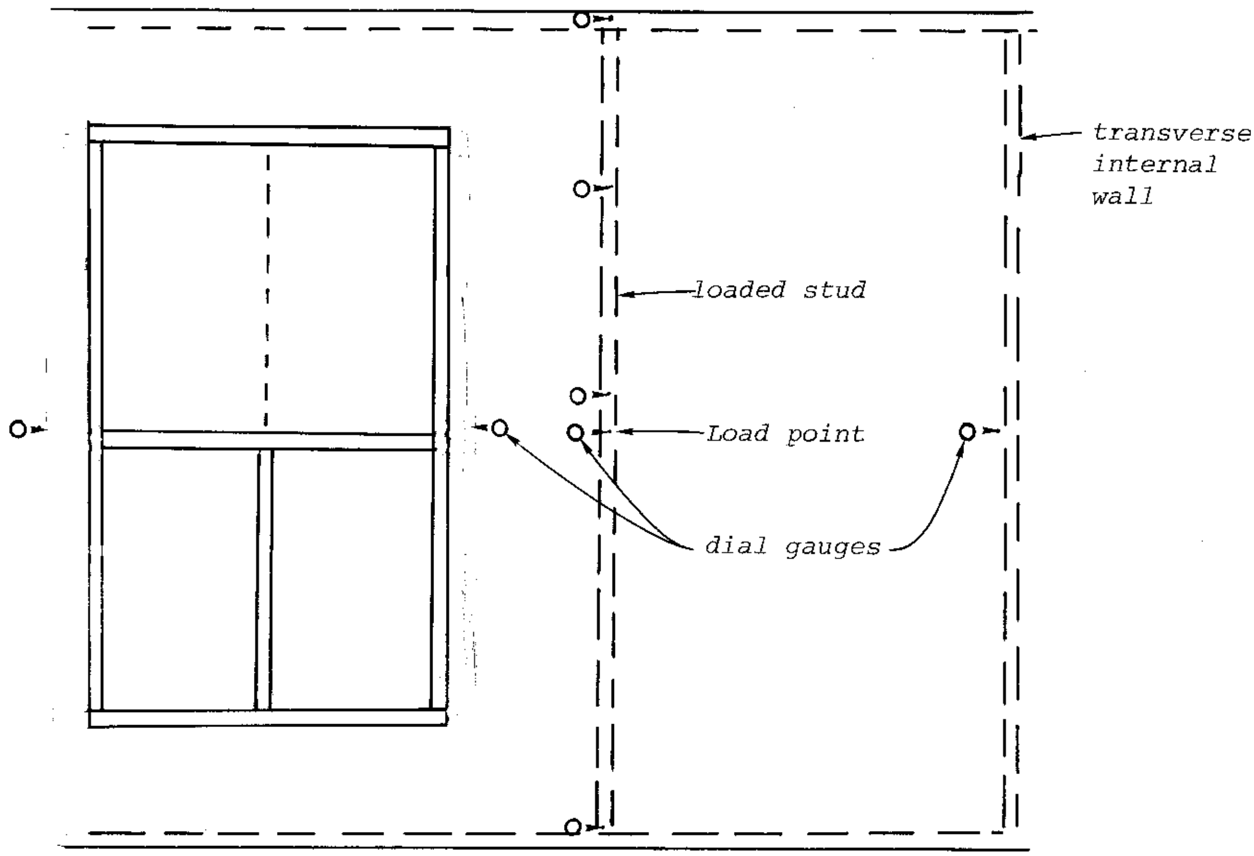


Figure A.4 Test configuration stud test

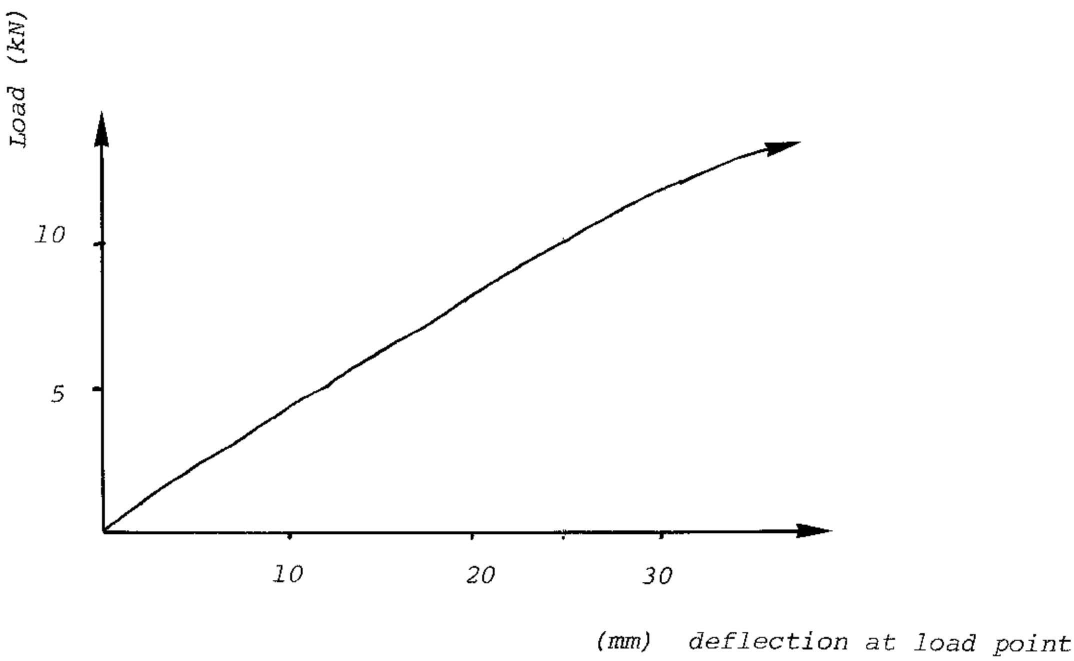


Figure A.5 Load-deflection curve for stud tests

Test History

The load was applied in 1 kN increments to 12 kN when gauges were removed. Again deflection of the loaded stud caused bending of the weatherboards and deflection of adjacent studs.

Failure commenced with the loaded stud moving away from the weatherboards in the vicinity of the load at 7 kN load. The ultimate load of 13 kN corresponded to the failure of the loaded stud in flexure at the load point. Again failure resulted in excessive deflection causing damage to internal linings and isolating the stud from the weatherboards, but causing little distress to the nails securing the stud to the top and bottom wall plates.

A.5 Stud Test 3

Geometry of Test

In this test, a load was applied at mid height of a single wall stud in an internal wall (Point C of Figure A.1). Noggings, plywood sheeting and cover strips were all left intact. Deflection gauges were placed at four points on the length of the stud to give an indication of the deflected shape, and other gauges were placed on adjacent studs and at the ends of the wall tested to ascertain the horizontal distribution of load to other studs in the wall.

Test History

The load was applied in 1 kN increments to 10 kN when the gauges were removed. The deflection of the loaded stud caused extensive buckling of the plywood sheeting and deflection of adjacent studs.

Failure commenced with the tearing of the plywood sheeting on the tension side at approximately 13 kN. Eventually, the loaded stud failed in flexure at the loading point at 14 kN.

A.6 Stud Test 4

Geometry of Test

In this test, a load was applied at mid height of a single wall stud in an external wall, in a similar manner to that indicated for Stud Test 1 (point D in Figure A.1). However in this case, all internal and external claddings and noggings were removed. Deflection gauges were placed along the length of the stud and on top and bottom plates adjacent to the stud.

Test History

At a load of 0.7 kN the stud started to crack at a check out for a diagonal brace, 600 mm below the load point. Failure occurred at a load of 0.95 kN by flexural failure of the stud at the check out. There was little deflection of top and bottom plates and the end driven nails securing the stud to both top and bottom plates continued to hold, inspite of some withdrawal.

The loads sustained in this test are subject to some doubt. Flexure tests performed on timber taken from adjacent studs indicate that the actual failure load could have been much larger than the figure obtained. As the securing nails at the top and bottom of the stud appeared to be much the same in test Stud 5, it is likely that an equipment manlfunction gave low load readings for this test.

A.7 Stud Test 5

Geometry of Test

This test, at load point E in Figure A.1, was identical to Stud Test 4, except that the stud did not have a check out for diagonal brace. The gauge locations were the same, and all claddings and noggings had been removed.

Test History

The load was applied in 0.2 kN increments at first to a load of 2.5 kN. At this load cracking of the stud in the vicinity of the load point first occurred. The load was then steadily increased to the failure load of

6.9 kN. At failure the stud was broken in flexure on both sides of the loading plate. At the failure load the two end driven nails securing the stud to top and bottom plates were still performing adequately although significant withdrawal had occurred, due to the deflected shape of the studs.

The loads sustained during the early part of this test, during which deflection readings were taken, are also subject to doubt. After the dial gauges were removed, a step increase in the load occurred, and the load at failure was comparable with specimen tests of stud timber. However, the loads obtained during the time that deflections were monitored appear to be much lower than those expected. It is presumed that an equipment malfunction took place prior to the step increase in load, and that all loads recorded at this time are suspect.

A.8 Roof Test

Geometry of Test

In this test, an uplift load was applied to the roof structure by two loading frames simultaneously. Load spreaders were used to apply equal loads to 12 points centred on points L and M in Figure A.1. The loads were applied to the underside of the purlins rather than the actual roof sheeting, and the deflections of the purlins relative to the top chords of the trusses were monitored at 4 locations. Figure A.6 shows a schematic diagram of the load application points and the purlin displacement measurement.

Test History

The load was applied in approximately 4 kN increment. At about 40 kN total load, the struts in the trusses had pulled away from the bottom chord. On relaxation of the load this displacement remained. The loading was continued until failure occurred, by the detachment of the purlins from the nailing blocks on the top plates. This damage is depicted in Figure A.7. A load versus displacement curve for the purlin top chord joint is also given in Figure A.7. The failure was quite sudden with 14 purlin to top chord joints failing simultaneously.

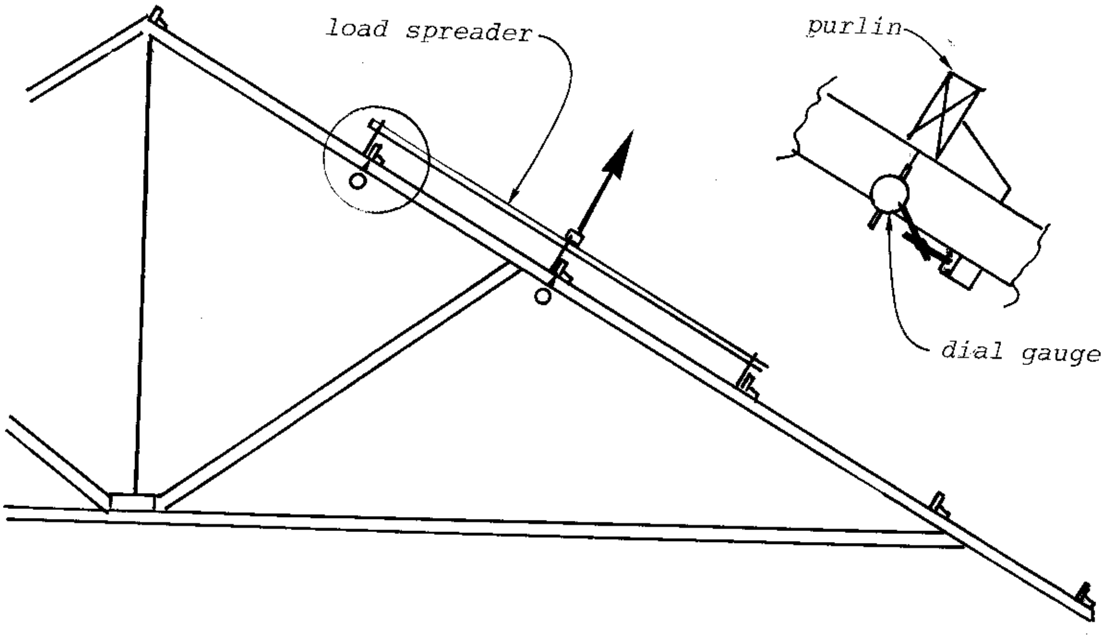


Figure A.6 Test configuration - Uplift test

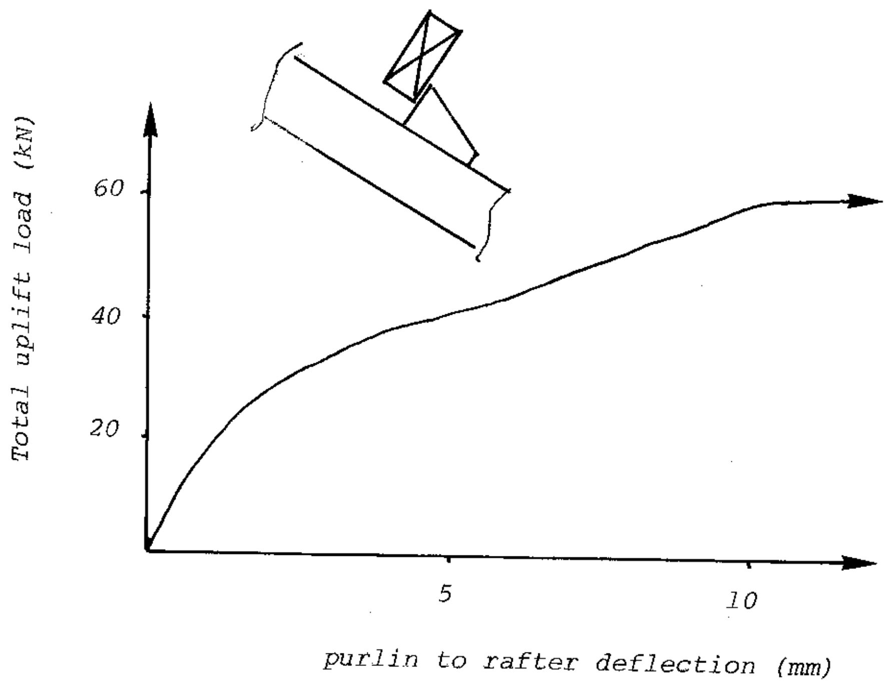


Figure A.7 Load-deflection curve for uplift test

A.9 Floor Test

Geometry of Test

In this test, a lateral load was applied to the floor structure of the house at points J and K in Figure A.1. The load was applied to the ends of floor joists and deflections of the outside bearers measured at five locations as shown in Figure A.8. These deflection measurements enabled the stiffness of the floor diaphragm acting as a cantilever to be determined.

Test History

At low loads, the deflection measurement points away from the loaded joists actually showed deflection in the opposite direction to the applied load. This was due to pivotting of the floor on stumps between the load points and measurement points. However between 40 kN total applied load and 60 kN total applied load, all deflection measurement points showed deflections parallel to the applied load and indicated that at most of the stumps, the bearers were sliding on the top of the stumps. At 85.4 kN total applied load, the movement became very pronounced, and the test terminated to prevent serious damage to the house by dragging it off the stumps. At this point the total permanent deflection amounted to over 50 mm for the portion of the house which experienced the lateral load. This occurred by either sliding of the bearers over the ant caps on top of the stumps or sliding of the ant caps over the stumps. In two locations, the bearers did not move significantly relative to the ant cap stumps but the whole stump rotated. The load versus deflection curve shown plotted in Figure A.9 is that of a dynamic friction failure.

A.10 Top Plate Tests 1 to 4

Test Geometry

In these four tests, a horizontal load was applied normal to the wall top plate at each of locations F, G, K and I as shown on Figure A.1. For each test, dial gauges were placed on the top plate, the roof frame and the roof sheeting along the loaded wall, so that deflections would give an indication as to the paths that the lateral forces followed in order to be transmitted to ground.

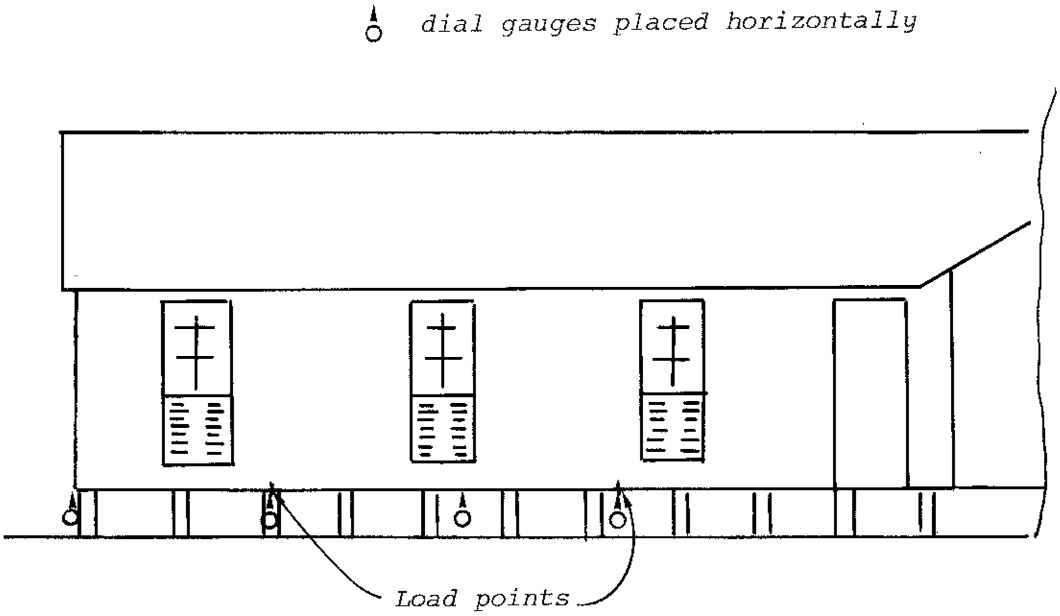


Figure A.8 Test configuration - Load applied at bottom of wall

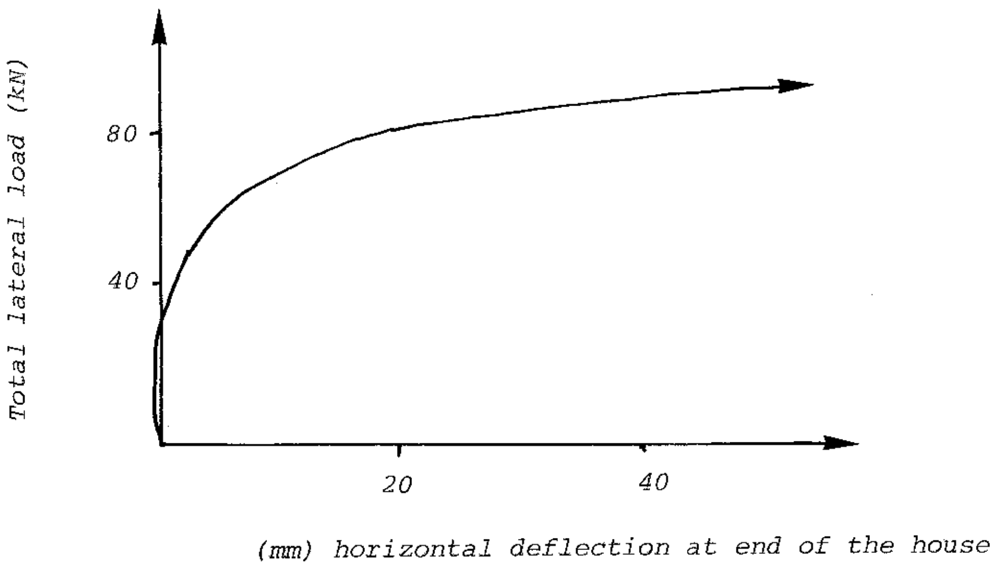


Figure A.9 Load-deflection curve for load applied at the bottom of the wall

Test History

During the loading in each of the four tests, the top plate bent inwards quite noticeably and ceiling panels near the load point rotated in plane. In two of the tests, ceiling coverstrips buckled in compression, and in all of the tests creaking noises were heard from the roof and side walls. Again, in all of the tests, loads near 50 kN could be sustained with little deviation of most gauges from a linear elastic load-deflection curve.

A.11 Top Plate Test 5

Test Geometry

A horizontal load was applied normal to the wall top plate simultaneously at locations F, G, H and I as shown in Figure A.1.

Test History

During the loading, the top plate bent inwards quite noticeably, particularly at load point F. The deflection of the top plate at load points G, H and I was limited by the proximity of internal walls that were parallel to the load direction. At a load of approximately 30 kN, the ceiling in line with load point F showed signs of severe deformation. This included the opening up of some cracks first seen in test Top Plate 1, the springing of ceiling cover strips and the opening of some new cracks in line with the applied load but on the other side of the house from it (a distance of 6 m from the load point).

At a load of 48 kN the top plate and approximately 2 m^2 of ceiling failed immediately adjacent to load point F. The top plate had been notched adjacent to the load point and failure occurred at the notch.

Load was then reapplied to load points G, H and I to 48 kN and the load at G increased to 65 kN with no noticeable damage to the structure. At this point the floor had lifted off the stumps in the vicinity of load point G.

A.12 Top Plate Tests 6 to 11

Test Geometry

These tests constituted a series of tests with a single lateral load applied normal to the wall top plate at location G as shown in Figure A.1. The maximum deflection of the load point was restricted to 20 mm to ensure that no permanent damage to the house structure occurred. Deflections were monitored at the same locations in all tests. These locations are indicated in Figure A.10.

The tests were separated by a modification to the structure which involved the removal of potential bracing elements from the house. The program was as set out below.

Test Top Plate 6

Removal of weatherboards from Northern wall

Test Top Plate 7

Removal of ceiling sheeting

Test Top Plate 8

Removal of roof sheeting from the southern side of the house

Test Top Plate 9

Removal of roof sheeting from the northern side of the house

Test Top Plate 10

Removal of all ceiling battens

Test Top Plate 11

The dial gauges were located as shown in Figure A.10 for the six tests, and a typical load-deflection plot is shown in Figure A.11.

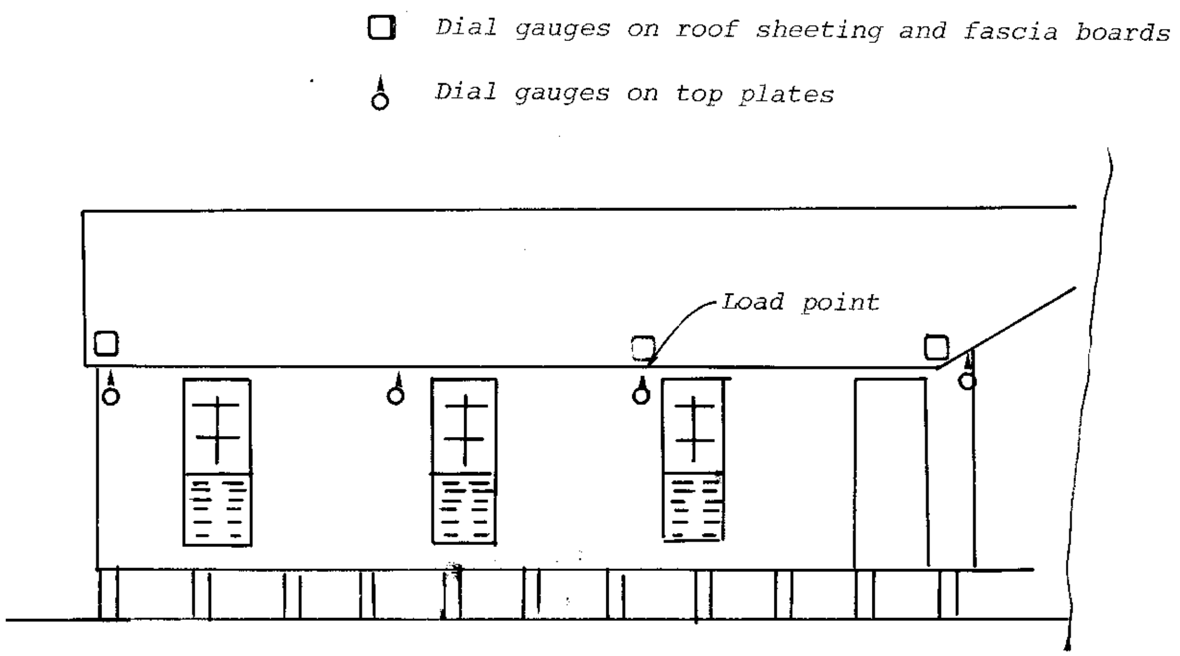


Figure A.10 Test configuration - stiffness tests with load applied to top plate

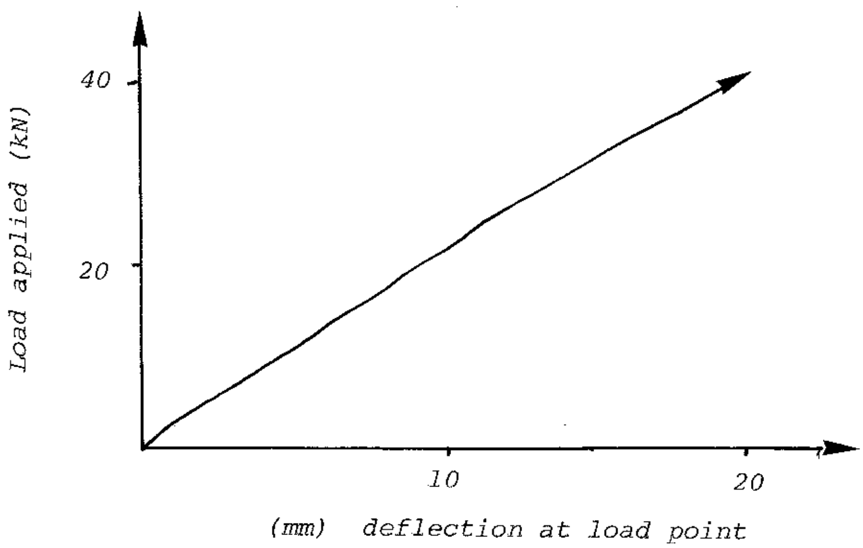


Figure A.11 Load-deflection curve for stiffness tests shown in Figure A.10.

A.13 Top Plate Test 12

Test Geometry

This test was performed on the internal wall that was immediately adjacent to lead point G as shown in Figure A.1. The wall had one side clad with A.C. sheets and the other with plywood and was removed from the house and tested in the laboratory. Both ends were held down to the test bed with steel rods and the load applied at top plate level in the plane of the wall. Four dial gauges were used to determine movement of the wall, and movement of the test bed. The test configuration and the location of the dial gauges is shown in Figure A.12.

Test History

The wall was loaded in increments of approximately 2.5 kN to 12.5 kN. This was approximately the same load sustained by the wall in the stiffness tests - Top Plate Tests 6 to 11. The wall was then unloaded and the small amount of permanent deformation indicated that the approximation of elastic behaviour in the stiffness tests was valid. Failure occurred at 28 kN and was precipitated by the failure of fasteners at the top of the A.C. panelling buckling of the plywood panelling in compression regions and tearing of the plywood fasteners in tension regions. Figure A.13 shows the load versus deflection curve for the panel.

A.14 Top Plate Test 15

Test Geometry

This test was performed on a section of ceiling complete with cover strips battens and joints as removed intact from the house. The panel measured approximately 3 m x 3 m and was tested in the manner detailed in Walker, Boughton and Gonano (1982). The appearance of the test configuration was similar to that shown in Figure A.12.

Test History

The load was applied in increments of approximately 0.3 kN to a load of 1.2 kN. This load gave a fastener stress level equivalent to that expected

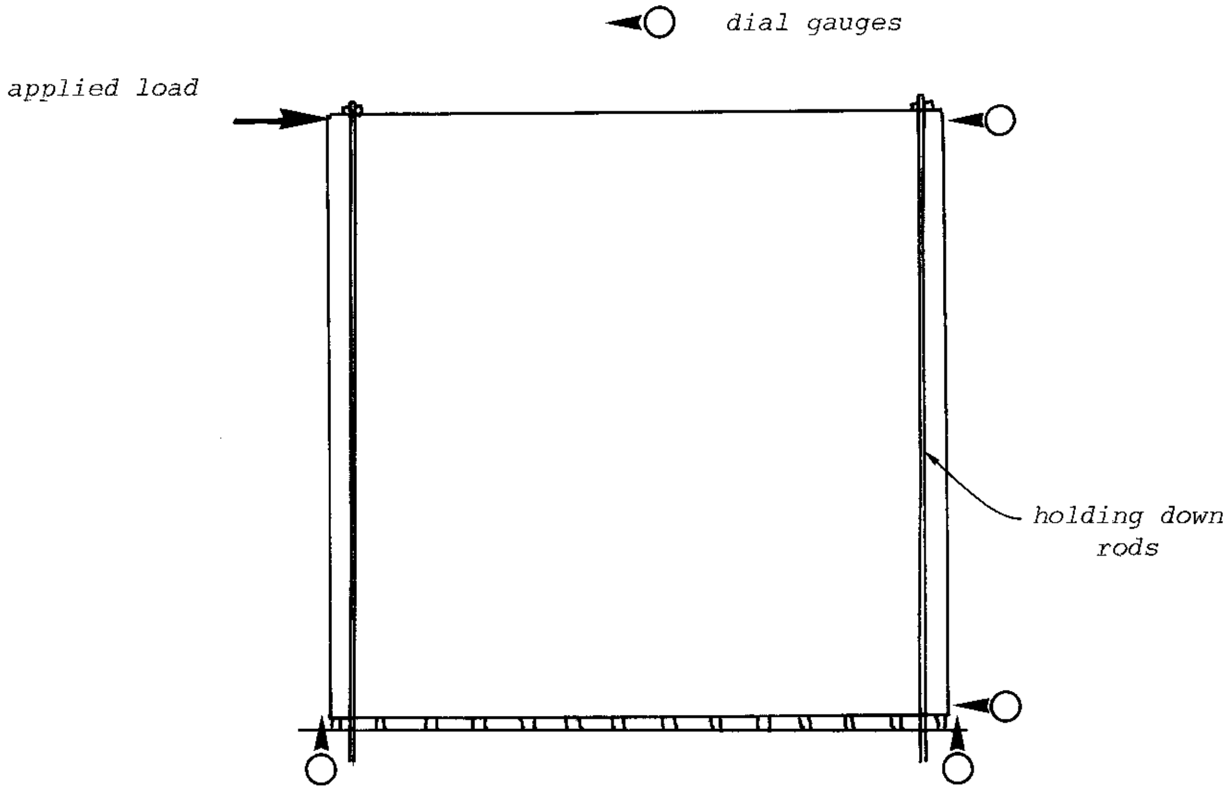


Figure A.12 Test configuration - Top plate test 12

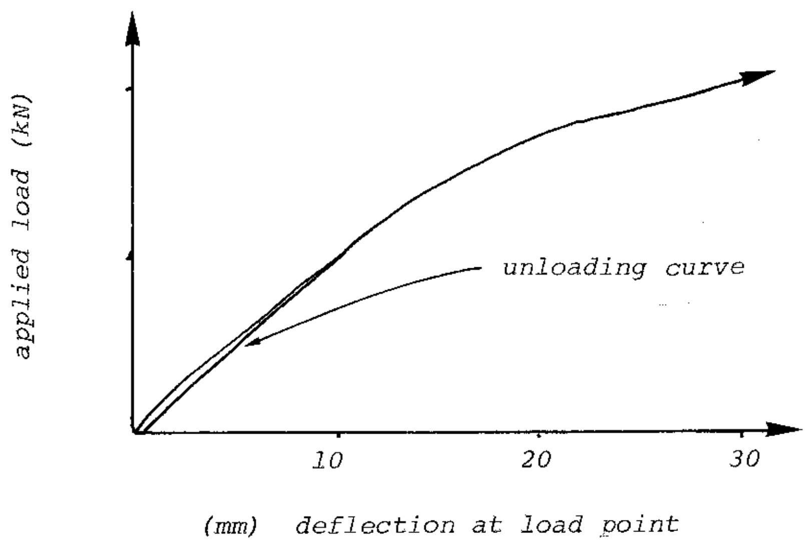


Figure A.13 Load-deflection curve - Top plate test 12 of the loads applied in tests Top Plate 2, 6 and 7. At this load some signs of distress were obvious near corner fasteners but unloading and subsequent reloading showed little deviation from elastic behaviour. Loading then continued until a failure occurred at 8.5 kN. The failure was due to tearing of the cane-ite sheeting at tension corners and the pull through of some edge fasteners.