CYCLONE TESTING STATION

SIMULATED WIND LOAD TESTS ON THE TONGAN HURRICANE HOUSE

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Technical Report No. 23

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SUMMARY

Simulated wind load tests were conducted on a Tongan hurricane house, designed by the Tongan Ministry of Works and mass produced in Tonga for replacement housing following extensive damage caused by cyclone Isaac in 1982. The prefabricated house kit was chosen at random from those on the production line in Tonga and assembled under the direction of a Tongan supervisor to ensure it was typical of those already built in Tonga.

During the cyclic loading sequence which simulated the buffeting action of a cyclone, a number of localised failures were observed. Of these, the failure of truss tie down straps would have caused significant failure of the complete house. A new truss tie down detail was designed, tested in the laboratory and incorporated into the house. The modified house was then able to endure the full complement of cycles based on the Tongan Design Loads.

After having tested the modified house to destruction some comments on the failure loads have been made in relation to the original Design Loads and also to loads obtained from the most recent Australian Wind Loading Code and a wind tunnel study on a model of the hurricane house. Other recommendations have been made on the suitability of the house for extension and its performance in sites with varying protection.

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1.   INTRODUCTION

In March 1982 cyclone Isaac devastated much of the tiny Pacific island
kingdom of Tonga. Extensive damage was caused to buildings and crops.
On some islands in the central Ha'apai group more than 90% of houses
were damaged, many beyond repair. Detailed reports of the damage caused
to buildings, and the impact on the society and economy have been
published elsewhere (Reardon and Oliver 1982, 1983; Oliver and Reardon
1982).

On the main island, Tongatapu, where about two thirds of the population
live, there was extensive damage to villages on the north west peninsula
but not very much damage at Nuku'alofa, the capital. This was fortunate
as it allowed post disaster operations to function with reasonable
efficiency. One such operation was the distribution of tents to alleviate
the immediate problem of some 2000 houses destroyed.

Tent villages were established to solve the short term accommodation
problem. The Tongan Ministry of Works (MOW) then developed a reconstruction
programme whereby they planned to build approximately 2000 new houses plus
necessary school classrooms within a period of about two years.

2.   THE TONGAN 'HURRICANE HOUSE' PROGRAMME

2.1   Design and Fabrication

It was obvious after cyclone Isaac had destroyed so many of the 'European
style' houses in Tonga that it would be a folly to rebuild using the same
construction techniques. There was a need for engineering input into the
reconstruction programme to ensure that future cyclones of intensity
similar to Isaac would cause little or no damage. Estimates of wind
speed in cyclone Isaac indicated that it was by no means an extraordinary
event.

Although the MOW had qualified engineers capable of making a classical
structural design of a cyclone resistant building, they lacked
experience in designing houses and utilizing some of the complex inter-
actions that can occur in domestic construction. Recognizing this fact,

the MOW sought assistance from the Building Research Establishment, U.K. (BRE). Like the Cyclone Testing Station, BRE had visited Tonga immediately after cyclone Isaac and had made some preliminary recommendations at that stage. Further,Dr. Keith Eaton of BRE, who acted as advisor, had been involved in a similar exercise at St. Vincent in the Caribbean (Eaton, 1979). The result of the combined MOW, BRE design became known as the 'hurricane house'.

Although blockwork houses generally performed better during cyclone Isaac than did timber framed ones, the latter form of construction was chosen for the hurricane house. The concept of the system was to prefabricate wall panels and roof trusses in a factory at Nuku'alofa and transport the components complete with nails,bolts, cement and even aggregate to the villages or the islands for erection.

rw   panel with window right
lw   panel with window left
rwd rw panel with door
lwd lw panel with door
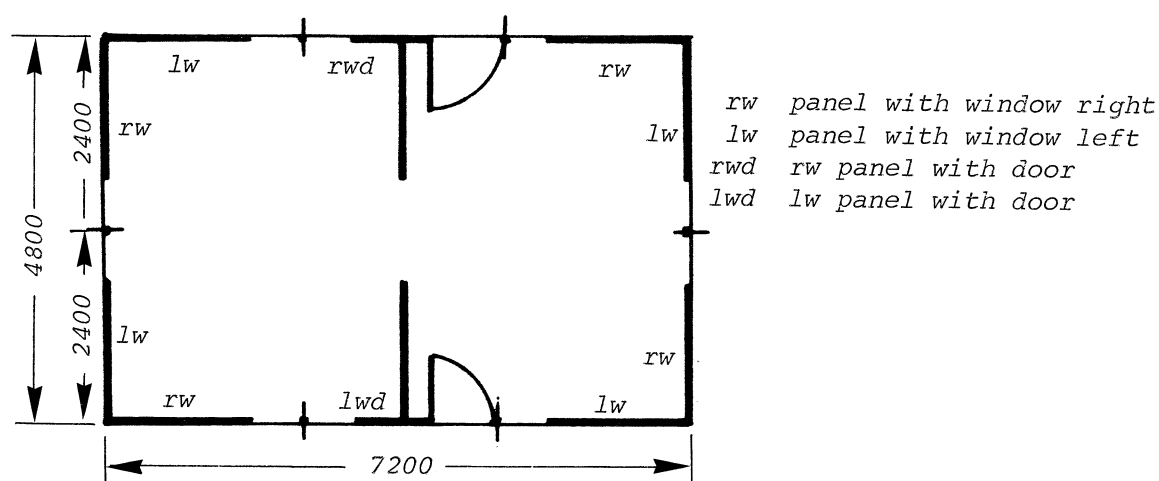
Figure 1   Floor Plan of Tongan
            Hurricane House

A standard floor plan, Figure 1, was decided upon as suitable for the basic needs of a Tongan family. Traditionally,cooking and ablutions are done in outhouses. The dimensions were chosen as multiples of the standard 2.4 m panel width, with wall height also 2.4 m.

Building materials for the reconstruction were purchased with overseas aid money. Jigs were set up in a factory so that standard wall panels could be

made. There were two types, one having a 1.8 m length of clad wall then 0.6 m of window opening, and the other having a door opening as well as a window. Both types of standard panel were made either left or right handed. Thus they could be assembled to form a 1.2 window opening. External cladding of either plywood or fibre cement sheeting was fastened to the frames in the factory. There was no internal lining.

The roof trusses were fabricated in a factory, and used hammer-in type toothed plate connectors at the joints. A standard roof pitch of $22\frac{1}{2}°$ was used, and the trusses incorporated a 300 mm eaves overhang.
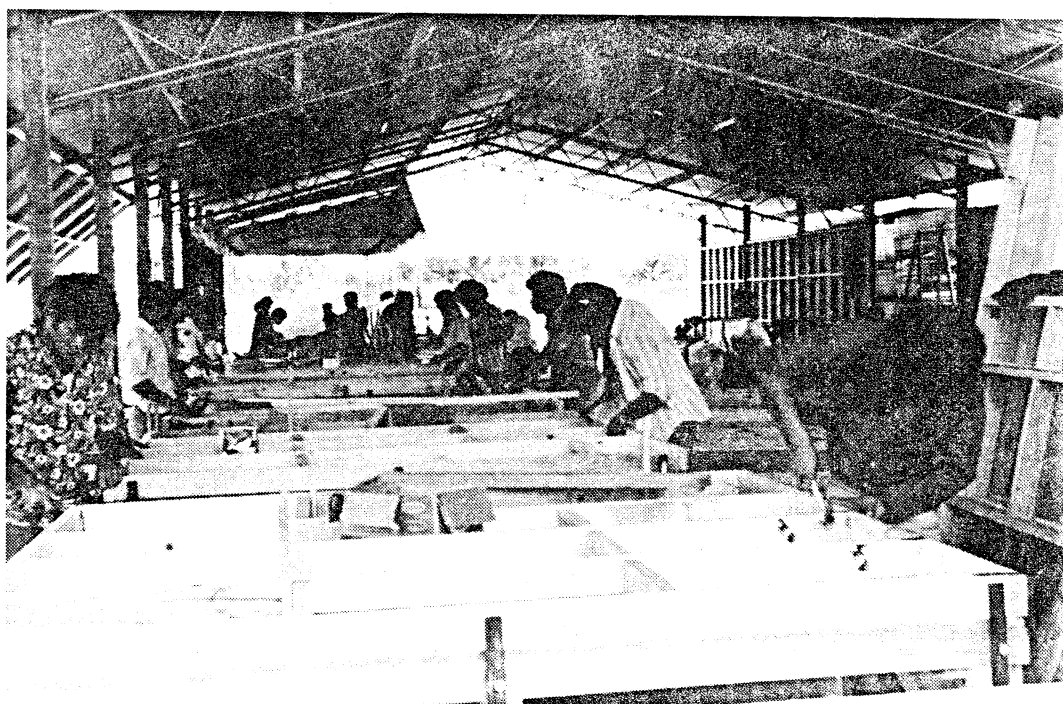
Figure 2 Factory production

The concept of prefabricating panelized components proved to be quite successful. Jigs were set up and used to ensure uniformity of the products, Figure 2. The simplicity of the design meant that only a few variations were needed. Some 47 men were employed in the fabrication yard and by the time they had fully established their routine, they were producing components for 50 houses per week. The components were then transported either by road to villages on Tongatapu or by sea to other islands. Erection of the components took a gang of five men about 3 days. The gang usually consisted of 2 MOW men and local self help groups. The first few houses were erected on concrete slab floors, but because of cost and local

preference the system was changed to suspended wooden floors.

The total cost of a hurricane house is about T$2900. The people who had their houses destroyed in the cyclone were offered one for T$700, the balance being supplied by overseas aid agencies. To put these figures somewhat into perspective, the men fabricating the house components earn about T$3 per 10 hour day.

Figure 3 shows a hurricane house.

Figure 3   Completed house

2.2   The Test House

The Building Research Establishment was aware that the Cyclone Testing Station had the facilities and the ongoing research programme to conduct simulated wind load tests on full scale houses. It therefore suggested to the MOW and the Tongan Government that it would be in their interest to have a prototype house tested in Australia. By so doing the strength of the house under simulated cyclone conditions could be determined. Further if there were any unforseen weaknesses incorporated in the design they

could be pinpointed during the test programme. Modifications could then be made and tested, and if successful could be incorporated into existing houses. Hence any weakness could be rectified before the advent of the next cyclone, thereby representing a significant cost saving to the community.

The Tongan Government agreed to BRE's suggestion and made a set of house components available for shipment to Australia. Co-operation in the exercise was sought and gained from a number of other governments. The British Government funded the transportation costs from Tonga to Townsville. The New Zealand Government paid the fares of Mr. Pilimi 'Aho, Assistant Secretary of Works Tonga, to supervise and participate in construction of the test house. The Australian Government through its Australian Development Assistance Bureau funded part of the test programme.

Having Mr. 'Aho come to Australia to supervise construction meant that the house would be built exactly to Tongan standards, and would thus represent as closely as possible a typical hurricane house. Only the ground conditions would be different.

Structural details of the test house were nominally as follows:

    200 mm dia. pile stumps on 1200 mm grid
    100 x 75 mm bearers at 1200 mm spacing
    100 x 50 mm floor joists at 600 mm spacing
    100 x 50 mm wall studs at 600 mm spacing
    100 x 50 mm noggings - 2 rows
    100 x 50 mm top and bottom plates
    4.8 m span roof trusses at 1200 mm spacing
    75 x 50 mm battens at approximately 600 mm spacing

The framing timber was from the USA. It was branded hem-fir, was dressed, arrised and measured 88 x 39 mm on average.

The house was clad in 8 mm external plywood, and had no internal lining or ceiling. The internal partition wall was basically non-structural. It was clad with 4.5 mm hardboard fastened with light gauge brads at about 150 mm spacing.

The trusses supporting the corrugated steel roofing were located directly over wall studs and were fastened to them with perforated light gauge metal strap, 25 mm wide and 1.2 mm thick. Joints between studs and plates were made with one framing anchor located on the internal face. The plywood extended from plate to plate on the outer face of the wall. The bottom plates were bolted to the subfloor timbers with M12 bolts at 1200 mm spacing.

The geometry of the system meant that every second truss was located over a joint between panels, thus it was supported by two studs and strapped down to them. Adjacent panels were secured together by three M12 bolts through the end studs.

Figure 4 shows the house ready for testing.

Figure 4   Test house

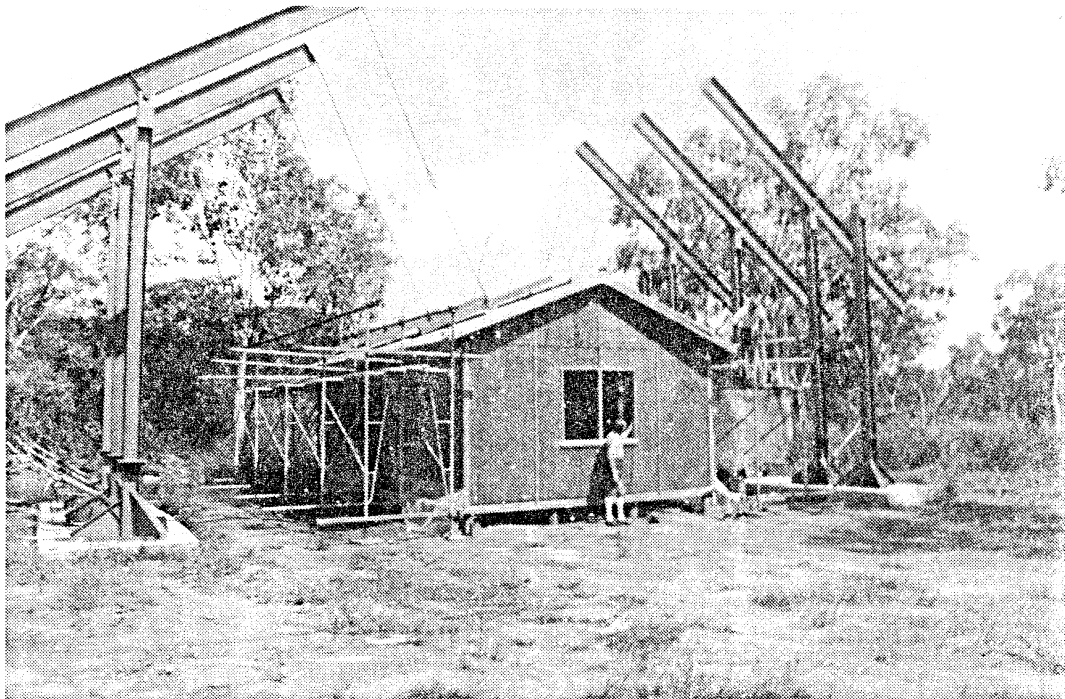3.   LOADING AND INSTRUMENTATION

High speed air moving past a house generates uplift loads on the roof and drag and suction loads on the walls. These also combine to place on over-turning moment on the house. The aerodynamics of the wind/house interaction are well documented by Holmes (1980). In the tests described in this publication, the effect of the wind was simulated by directly applying

lateral and uplift loads to the house. The response of the house was
determined by recording deflections as the loads were applied.

3.1   Loading System

The loading system was designed to simultaneously place uplift and lateral
loads on the house. The lateral loads acting on the building consist   of
pressure loads normal to the windward walls and suction normal to the leeward
walls. There are also lateral components of the internal pressure which have
no net effect on the house as a whole, and the suction forces on the side
walls also cancel.   The net lateral load on the house is therefore dis-
tributed over the long sides of house. The net uplift on the roof structure
consists of aerodynamically induced suction on the top surfaces of the
roof and internal pressure acting upwards on the underside of the roof
sheeting.

For the purpose of testing, lateral forces were placed on the house at the
top of the wall. These forces were equivalent to one half of the total
lateral wind load, assuming that half of the lateral forces are carried to
the top of the walls and the other half to the bottom of the wall. Wind
tunnel pressure distribution graphs show that for a house with an elevated
floor, the errors introduced by this assumption are negligible. The full
lateral load was applied to only one side of the house, whereas often in
reality much of the suction would have been placed on the leeward side
and most of the pressure on the windward side. However, it is possible
to envisage the full lateral load being placed on the windward wall with
some combinations of window or door openings giving rise to appropriate
internal pressures.

The lateral loads were applied using the apparatus shown in Figure 5. The
forces were generated by a hydraulic ram (a) which applied tension to a
cable. This cable passed over a pulley (b) and through the house to a
load spreader (c). Three such loading frames, each equipped with spreaders,
distributed the lateral loads to six points at 1.2 metre spacing over the
width of the house. The wall subjected to this lateral load will be known
as the windward wall throughout the remainder of this report. The same
loading frames were used to apply point loads immediately adjacent to the
three transverse walls.

The uplift forces were applied to the entire roof of the building using
a total of 6 hydraulic rams, loading frames and load spreaders. The total
uplift force, consisting of the sum of internal pressures and external
suction was applied to the underside of the roof sheeting using a config-
uration also illustrated in Figure 5.
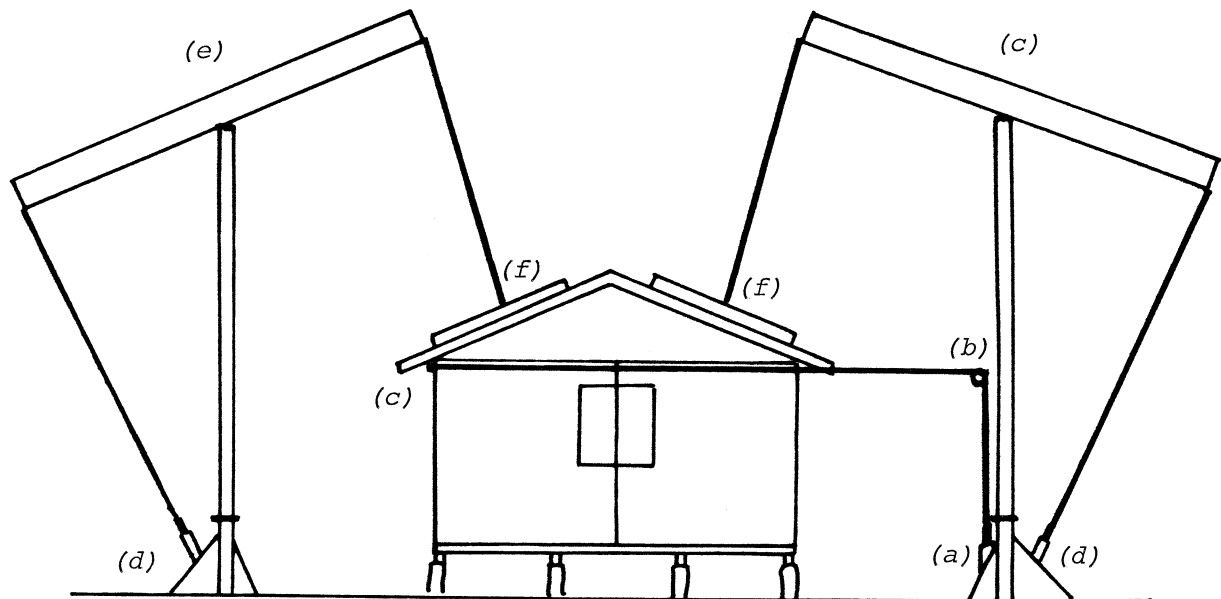
Figure 5   Configuration of Loading system

In this case the hydraulic rams (d) pulled downwards on one end of large
"see-saw" beams (e). The other end of these beams lifted the load spreaders
(f) which distributed the uplift loads evenly to the roofing over a total
area of 43 m^2. The applied loads were perpendicular to the plane of the
roof on each side of the ridge. The load spreaders consisted of a large
number of interconnected steel beams, loaded in the centre and carrying
load at each end. The load spreaders had 48 load rods passing through
the roof sheeting to eight timber beams pulling upwards on the underside of
the roof sheeting. These timber loading devices were adjacent to the
purlins, and ensured that the purlins were loaded through the roofing screws.
The weight of the steel load spreaders was deducted from the measured
loads to give true uplift on the roof.

The loading equipment was used in three different applications. During
the construction phase, non-destructive tests were performed on the house

to determine force paths within the structure. The results of these tests are discussed in Section 5. After completion, the house was subjected to cyclic loading to simulate the prolonged action of cyclones as detailed in Section 3.3, and then loaded statically to destruction.

3.2   Load and Deflection Measurement

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

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

Deflections of the house were measured at 50 locations on the structure, with the datum provided by independent scaffolding at the front and back of the house. The deflection measuring transducers were fixed to this datum with magnetic bases, and their outputs were relayed to a micro computer for processing and storage. The system is depicted in Figure 6, and has been described in detail by Boughton (1983).

For the most of the tests, deflection transducers measuring lateral displacement were positioned at 1.2 m intervals along the top and bottom of the windward and leeward walls, and also along the bearer next to those walls. Other transducers were used to measure the upward displacement of the eaves on both windward and leeward sides of the house.

During the destructive testing programme, described in Section 6.2, the observation of uplift of the footings became important. The transducers that had previously monitored the lateral displacement of the floor, were moved to monitor the upward displacement of the floor.

The load and deflection data was stored on the micro computer and also transferred to magnetic tape. During the course of tests, a graph of deflections processed by any transducer could be plotted against the applied load. In this way, the micro computer was able to assist the test operator in the identification of structural components that had yielded.

lateral displacement
measurement

uplift
measurment

Positioning of Transducers

steel measurement plate
(accommodates lateral movement)

VRT displacement transducers

RHS transducer mounting
member

Mounting of Transducers

scaffolding
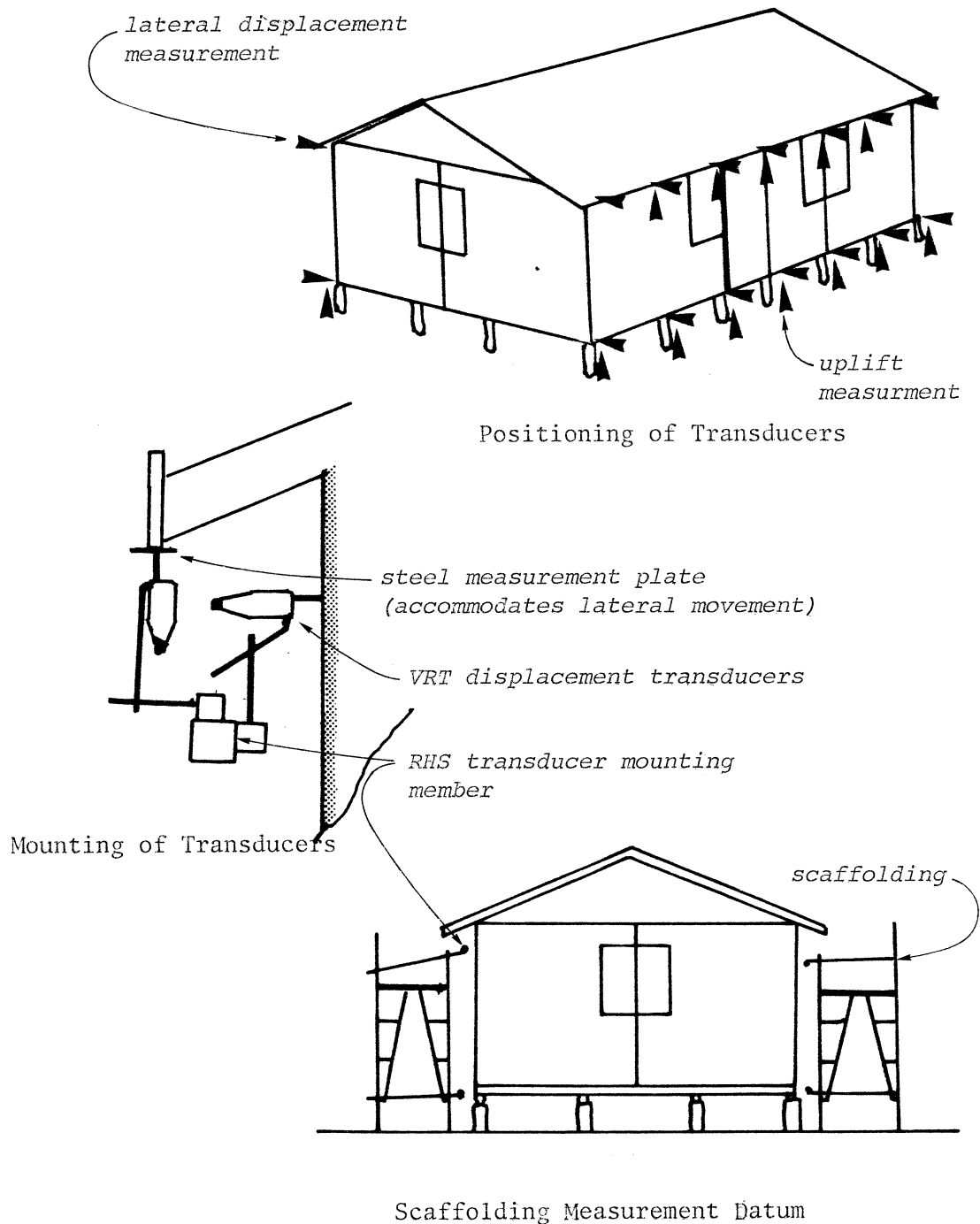
Scaffolding Measurement Datum

Figure 6 Deflection Measurement

3.3   Cyclic Loading

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

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

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

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

Lateral loads during a cyclone fluctuate less rapidly as they reflect the
turbulence in the air stream rather than the more severe structure induced
turbulence. Thus a smaller number of cycles was set out in Technical
Record 440 for wall loadings:

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

In order to accurately simulate the total effect of a cyclone on the house,
the uplift and lateral loadings were implemented simultaneously. Throughout
the sequence ten uplift cycles were required for each lateral load cycle. At
a signal from the computer, a hydraulic pump was started, and the loading of

both uplift and lateral force rams was commenced. Actually applied lateral
and uplift loads were separately monitored by load cells and each set of rams
was isolated when the appropriate maximum load was reached. The deflections
of the house were then read and the rams were unloaded. Another set of
readings was taken at the minimum load. The lateral load rams were then
isolated and nine uplift cycles were performed. For the tenth cycle the
lateral load rams were again engaged thus giving a sequence of 1 combined
load cycle, 9 uplift only cycles, 1 combined load cycle, ... until
the prescribed number of cycles had been completed. As most test involved
large numbers of cycles, deflections were read at selected times only
during the test. Typically 20 to 30 sets of deflection readings were taken
for each set of cycles.

4.   WIND LOADS ON THE HOUSE

The design of the Tongan hurricane house had been checked by a civil engineer,
Taumoepeau (1983), and the design sheets were used to determine the criteria
for loading. They show that loadings were based on the New Zealand Standard
NZS 4203:1976, and produced an eaves height wind speed of $62ms^{-1}$. This
figure is very similar to that obtained from AS 1170 Part 2 - 1983 for
cyclone prone areas and assuming terrain category 1 exposure. The assumption
of terrain category 1 as defined in the Australian code seems quite
reasonable as for many of the house sites, particularly on remote islands,
the houses will have positions with frontage onto the sea. In the light
of published information on cyclone generated maximum wind speeds associated
with cyclone Tracy (Walker, 1975), and cyclone Kathy (Boughton and Reardon,
1984) the $62ms^{-1}$ used as a design wind speed appears quite realistic.

The Australian code defines terrain category 1 as "exposed open terrain
with few or no obstructions and in which the average height of any objects
surrounding the structure is less than 1.5 m". A note states that the
category includes open seacoasts and flat treeless plains. The code defines
other degrees of exposure including terrain category 3, "terrain with
numerous closely spaced obstructions having the size of domestic houses".
This category will be referred to in Section 8.

4.1   Determination of Wind Loads using NZS 4203:1976

The Tongan design check was based on the New Zealand Standard with an assumed
internal pressure coefficient of +0.3. Due to the open eaves on the building
this figure seemed appropriate, as a reliable air path through all walls
would always be available. The external pressure coefficients are shown in
Figure 7, as are the total loads calculated from these pressures. The result-
ing net uplift pressure of 2.59 kPa for the roof was used as the basic design
load for the application of cyclic loading in uplift. The total equivalent
racking load at top plate height, 4.21 kN/m length of wall, was used as the
basic design load for the application of cyclic racking loads. As such
these loads are rightly referred to as the 'Design Loads'.

4.2   Determination of Wind Loads using AS 1170 Part 2 - 1983.

The most recent edition of the Australian Wind Loading Code, AS 1170 Part
2 - 1983 was released in December 1983, quite some time after Mr. Taumoepeau's
design check on the house. It is more liberal than previous editions. The
New Zealand Loading Code is based extensively on the Australian Standard, so
it is possible that the pressure coefficients as defined in the current
Australian Standard may be reflected in later editions of the New Zealand
Code. Although the new Australian code was not available for the design of
the Tongan hurricane house, an evaluation of the wind loads based on wind
speeds and pressure coefficients from that code will be made to provide an
interesting comparison with the Design Loads, and a possible benchmark for
future work in the South Pacific.

The internal pressure was calculated on the basis of having significant
wind penetration of the windward wall and leakage through the other walls.
This could be achieved by the opening of the windward door. Small
movements of door jambs will release many locks, and the lateral loads
on a door often prove too high for small screws on latches. This results
in a door opening at the height of a storm, allowing significant build up
of internal pressure. As this house was designed for beach front conditions,
and appropriate loads had been used, flying debris was not considered a
problem. However the leakage over unlined walls and around louvres and
doors in leeward and side walls would compensate for the loss of a few
louvre blades together with an open door in the windward wall. Under
these conditions, the ratio of the area of windward openings to all
other openings is approximately 1.5 and the appropriate net internal

pressure coefficient is also +0.3. For the case of broken or open windows
in the leeward wall, the internal pressure coefficient would be less.
The external pressure coefficients given by the standard and the total loads
calculated from these pressures are shown in Figure 7.

Due to the generally reduced external pressure coefficients given in the
current standard, the AS 1170 Part 2 - 1983 analysis gives significantly
lower loads than the actual Design Loads. While the AS 1170 loads are not
the design loads for this house, their recognition as the best current
estimation of wind loading on a house gives them credence in evaluating
the behaviour of the house in cyclone events.

The pressure coefficients given in the Australian code have largely been
derived from wind tunnel studies using point pressure measurement at
discrete locations over all external surfaces of the model. They are
therefore believed to be representative of the structural effect of the
wind on elements supporting relatively small areas of external surface.
This would include battens, batten fasteners, trusses, truss fasteners,
studs and top and bottom plates.

4.3   Determination of Wind Loads using a Wind Tunnel Model Study

In order to determine the total wind loads on the house, a small model
was constructed at 1/50 scale and mounted on a sensitive force balance
in a wind tunnel. The complete report on the wind tunnel tests, Jancauskas
(1984), describes the method in more detail. Using the force balance
method, only information on the total structure loads is available, with
the effect of internal pressure being ignored. The results of the wind
tunnel study are also shown in Figure 7, however because the data produced
is independent of the internal pressure, no estimate of the uplift on the
underside of the roof sheeting could be made.

The wind tunnel study did not give pressure distributions on the house
directly, but gave the total load effect of the wind on the house as a
horizontal force, a vertical force and an overturning moment. These
could be directly related to the total loads given in Figure 7. As each
total load is a combination of effects on a number of surfaces it is not
possible to determine either the proportion of load acting on any

particular surface or its pressure coefficient. Therefore no pressure
coefficients are given in Figure 7 for the results of the wind tunnel
study.

	Design loads from NZS	Other Wind Loads	
		from AS 1170 - part 2 - 1983	from wind tunnel study

pressure coefficients (C_p)

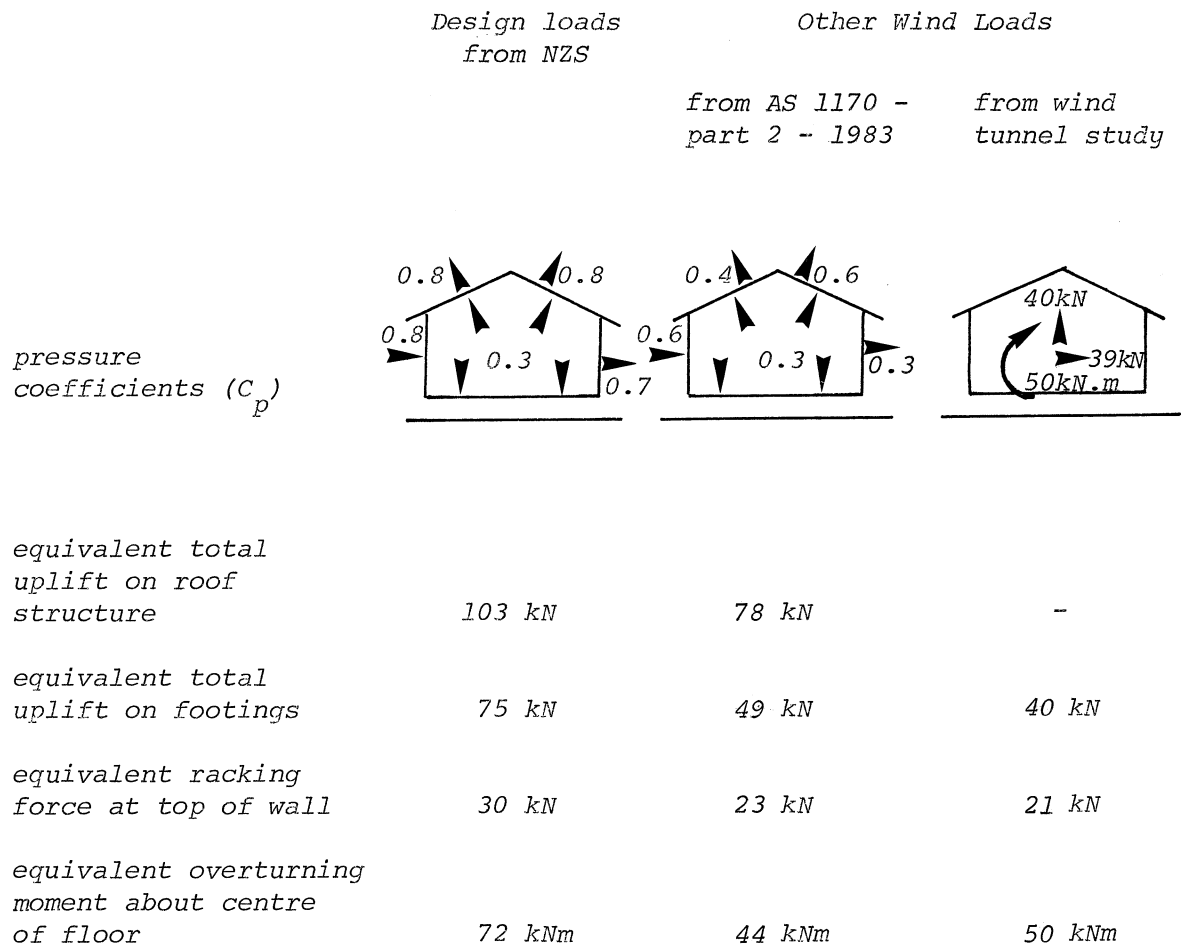|	Design loads from NZS	from AS 1170 - part 2 - 1983	from wind tunnel study
equivalent total uplift on roof structure	103 kN	78 kN	–
equivalent total uplift on footings	75 kN	49 kN	40 kN
equivalent racking force at top of wall	30 kN	23 kN	21 kN
equivalent overturning moment about centre of floor	72 kNm	44 kNm	50 kNm

Figure 7   Pressures and Loads on Tongan Hurricane House from
different Wind Load Analyses.

4.4   Comparison of Wind Loads Calculated Above

All of the loads calculated using AS 1170 Part 2 - 1983 were between 60%

and 80% of the Design Loads calculated using the New Zealand loading code.
This reduction reflects changes to pressure coefficients incorporated in
the latest edition of the Australian Standard, largely as a result of
many wind tunnel experiments on low rise buildings (Holmes 1980).
These wind tunnel tests used point pressure measurement to obtain accurate
measurements of loads on elemental areas of the models. AS 1170 Part 2 -
1983 could be said to represent the current state-of-the-art for load
determination for structural elements.

The wind tunnel study on the Tongan house yielded total loads on the
complete structure that were more likely to be representative of the
loads on the house footings in a design wind event. It is interesting
to note that the discrepency between the AS 1170 Part 2 - 1983 derived
total lateral and uplift loads and those calculated from the wind tunnel
model tests on the Tongan house amounted to less than 20%. The total
overturning moment was also similar to that calculated using AS 1170
Part 2 - 1983 loads. The similarity has been observed in other comparisons
between the two analysis methods. The model used for the total load study
had venting underneath the floor space as did the house tested. Other
studies have shown that this venting tends to reduce the total uplift
loads due to the venturi effect of air passing under the floor. The lateral
loads also tend to be reduced by the venting at the base of the wall.
Thus 'hurricane houses' built on a concrete slab on ground may be subjected
to marginally higher total loads than those shown in Figure 7.

When considering total loads on the whole building, for example footing
loads, the authors therefore consider that the wind tunnel study produces
the most realistic loads. For loads on structural elements within the
house, calculations based on AS 1170 Part 2 - 1983 represent the current
state-of-the-art and must be considered as the most realistic. However
the house was designed for a set of loads derived from another current
design loading code, and the tests were therefore based on these Design
Loads. In drawing conclusions from the test results the authors have
also used loading information from the latest Australian Wind Code and
wind tunnel study to assess house performance. These loads are summarised
in Table 1.

Table 1

Design Loads compared with Wind Loads from other sources

Load	Design Load (NZS 4203)	Wind Load from another source	Factor	Source
	D	W	D/W	
equivalent total uplift on roof structure	103 kN	78 kN	1.32	AS 1170
equivalent racking force at top of wall	30 kN	23 kN	1.30	AS 1170
equivalent total uplift on footings	75 kN	40 kN	1.88	wind tunnel
equivalent overturning moment about centre of house at ground	104 kNm	60 kNm	1.73	wind tunnel

5. LOAD TRANSFER MECHANISMS

During the course of construction, the house was subjected to some non-destructive lateral loads to determine load transfer mechanisms between the structural elements of the house. The relative simplicity of the house facilitated the interpretation of these results.

The lateral loads from the windward wall were traced into the internal partition and the two external end walls, where bracing action carried them to floor level. All of the stumps then functioned as vertical cantilevers to carry the loads from floor level to the ground.

The two near horizontal diaphragms in the house, the roof sheeting and the floor remained quite rigid at loads less than Design Load. During destructive testing, the floor showed little in-plane deflection even near the ultimate load of the house. However, the flexible connection between the top of the windward wall and the roof structure allowed the top plate to deflect significantly, and carry some load sideways in bending about its major axis. The majority of the lateral load at top plate level was transferred into the roof to be carried by that diaphragm to the end walls. During destructive testing, the roof diaphragm flexibility increased as roofing nails elongated the holes at sheet side laps.

This in turn transferred more load into the top plate in bending. Ultimately failure of a roof truss and subsequent detachment of the diaphragm grossly overloaded a top plate in bending, causing flexural failures.

This bending action had not been observed on the Hyne house (Boughton and Reardon 1983), where a more rigid joint between top plate and roof trusses was utilized. The flexibility of that connection in the Tongan hurricane house before modification allowed the top plate to move independently of the trusses before the straps took up the load. The modification detailed in Section 7 was performed after the non-destructive testing and also after some damage had been done to the truss/top plate connections. The bent shape of the top plate was probably built into the house by the modified roof tie down detail.

The transverse walls all behaved rigidly during the non-destructive tests. There was little movement of the windward wall relative to the leeward wall near the transverse walls, indicating a solid connection between the transverse walls and the windward and leeward walls. The internal wall was more flexible than the external walls,which was to be expected as it incorporated a large opening and very fine brads were used to nail the cladding to its frame. Its movement was 80% larger than the end wall movement for a given load, but that was not a large enough difference to cause it to attract less load during the application of uniformly distributed loads to the windward wall. It was only after the demise of the internal wall during the cyclic loading that loads from near the centre of the house were transferred to the end walls. Typical racking displacement of the walls at Design Load was 4 mm for end walls and 7 mm for the internal wall. These deflections were not sufficient to cause tearing of the sheets at the nails, and relative movement between sheathing panels in the walls could only just be noticed. The overall racking deflection was insignificant in comparison with the overturning deflections, and deflections of the top of the stumps.

Generally the house behaved as a rigid box at loads less than Design Loads. The overall pattern of movement was that of a stiff box being rolled about one edge. This is depicted in Figure 8.
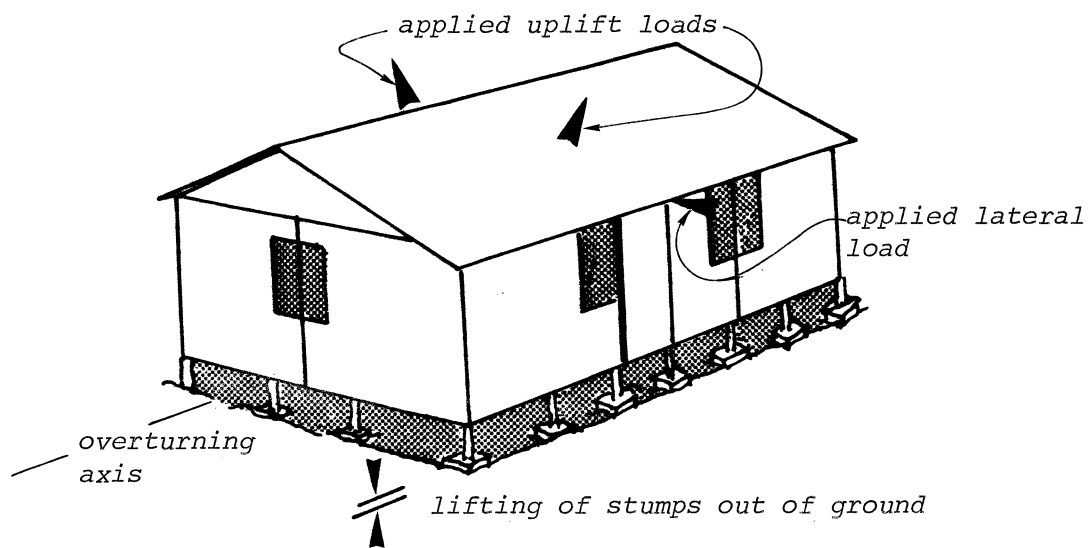
applied uplift loads

applied lateral load

overturning axis

lifting of stumps out of ground

Figure 8   Response of House to Applied Loads

6.   DESTRUCTIVE TESTING PROGRAMME

As detailed in Section 3.3, the house was tested for its capacity to resist repeated loadings. The entire house was subjected to simultaneous cyclic uplift loading and racking. After the successful completion of the entire cyclic loading sequence, the house was loaded to destruction by incrementing both lateral and uplift loads.

6.1   Cyclic Loading

6.1.1 5/8 Design Load cycles

In the prescribed loading pattern, cycles of 5/8 Design Load were applied. The requirement was for the house to resist 800 lateral load cycles and 8000 uplift cycles of this load. However after approximately 2500 uplift cycles (and hence 250 lateral load cycles) a failure was induced in a steel strap used to tie the trusses to the studs in the wall panels. The strap that failed had been installed so that the edge of the strap that

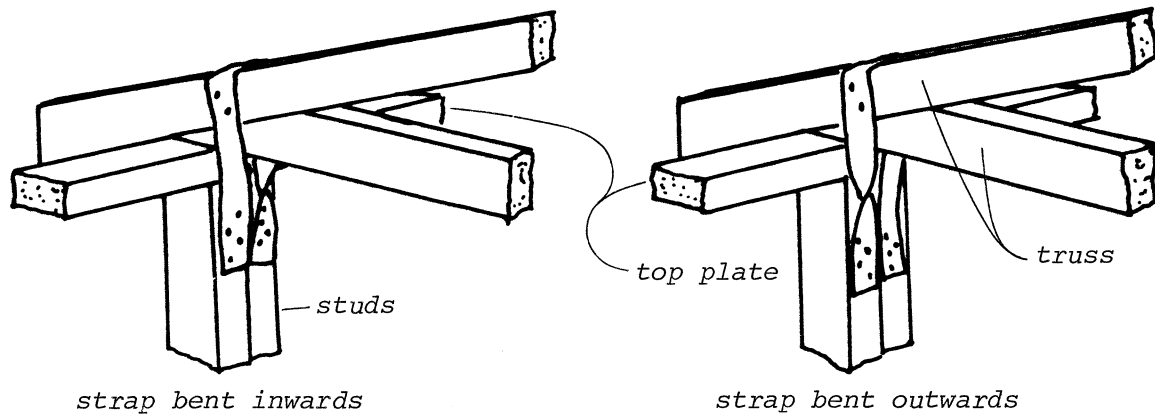strap bent inwards   strap bent outwards

Figure 9   Details of Truss Tie Down

was closest to the ridge on the roof truss was twisted outwards to be
nailed to the stud. All of the other straps had been installed so that
the edge closest to the ridge on the roof truss was twisted inwards at
the stud. This is illustrated in Figure 9. The outwards twist on the
strap that failed concentrated tensile forces at the edge of the strap
more severely than the inwards twist configuration, and this probably
led to a premature failure of that strap. At this stage it was noted
that where only three nails had been used on other straps, the nails
had started to work out of the timber. The inherent free play in the
joint allowed the strap to slide down on the shank of the nail on the
unload stroke, and then grip the shank on the upward stroke pulling the
nail out by a small amount with each cycle. The process is illustrated
in Figure 10.

The strap that had failed was replaced with one fixed in the same manner
as all the other straps and some extra nails were driven into other straps
to bring them all up to four nails per end of each strap. The test was
then resumed. After a total of 3720 roof cycles, a second tie failed in
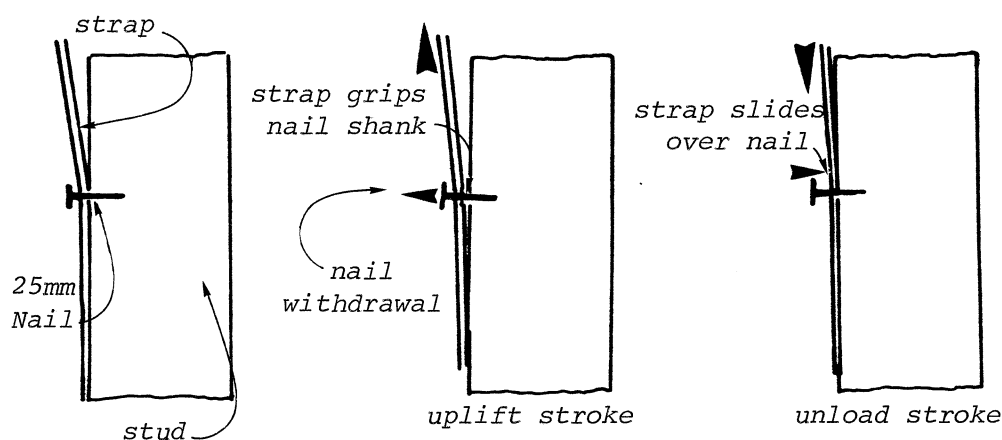a similar manner to the first. A crack in the strap metal was formed

Figure 10   Mechanism for withdrawal of Nails
from Truss Tie Down Straps

between the outside edge and a prepunched hole and slowly propagated across the width of the strap as shown in Figure 11. In this case the side that had not failed was capable of carrying the required load, so the test was continued. However, at a total of 4144 two more straps failed in very quick succession giving detachment of the roof on the windward side and resulting in significant lateral deformation of the top of that wall.

It was noted that of ten such truss tie down straps in the building four had failed due to crack propagation and of these three had been on the windward wall. The tie down detail on the windward wall, not only had to resist the uplift forces but also transmitted lateral load from the top of the wall to the roof structure, where diaphragm action of the roof sheeting served to spread it to the transverse walls. Reardon (1979) obtained failure loads of approximately 10 kN by nail pull out for similar straps with three nails per leg. Had four nails per leg been used the average failure load could have been as high as 13 kN. In the test house the uplift load at failure was 5.8 kN. The load in the strap due to lateral load was variable, but if the effect of the skew nails into the top plate and friction between truss and top plate was ignored

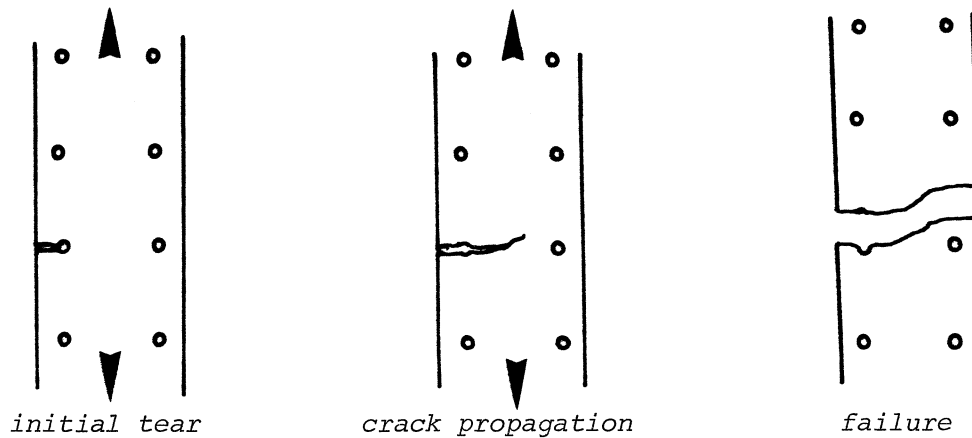*initial tear*   *crack propagation*   *failure*

Figure 11   Progressive Tearing of Straps

the additional strap tension could have been 2.4 kN. Thus the total strap load would have been between 5.8 kN and 8.2 kN depending on the effectiveness of other mechanisms for lateral load transfer between the walls and roof structure. Even so, this load was less than the expected failure load based on static test failure loads, and hence a failure at these lower loads indicated that the steel had deteriorated significantly with the application of load/unload cycles.

As a result of the systematic failure of the looped strap securing the trusses to the tops of the wall, all such straps were removed from the truss to wall joints and replaced with an alternative fastening which is detailed in Section 7. The house was then subjected to a new series of cyclic tests at 5/8 of Design Load. However, because more than half of the required cycles had been successfully endured by most elements within the house, it did not seem appropriate to subject them to the full number of cycles again. On the other hand, it was necessary to adequately test the new detail in context within the house. A compromise was struck whereby 8000 cycles of 5/8 Design Uplift Load would be applied simultaneously with

400 cycles of 5/8 Design Lateral Load. This gave the correct number of uplift cycles for the new tie down detail but did not subject the walls to much more than the correct number of cycles. This was achieved by implementing nineteen uplift cycles then one cycle with both uplift and lateral load simultaneously.
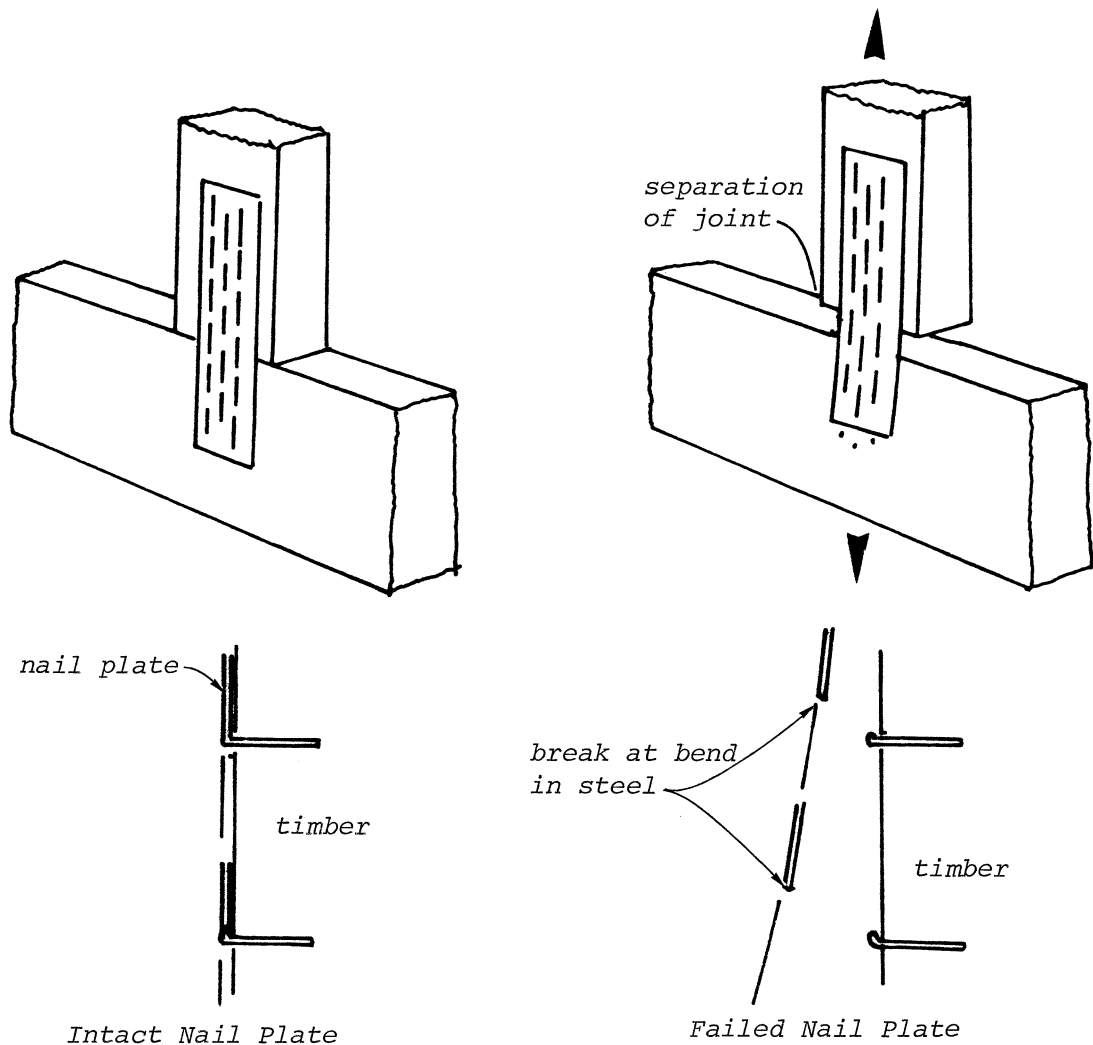

Figure 12   Failure of Nail Plate Teeth

After 5600 uplift cycles under these conditions, there were signs of structural distress in the vicinity of the internal wall. The toothed nail plates on each side of one of the short web members had failed by breaking of the teeth at the right angle bend as shown in Figure 12.

This joint acted in direct tension as the bottom chord of the truss was nailed directly to the top plate of the internal wall. In direct tension the bends in the steel nail plate tended to flex causing a stress reversal at that point. After some 5600 such stress reversals, metal fatigue had reduced the capacity of the teeth to carry load so that the entire joint had failed. The brads that secured the hardboard sheeting to the pine frame on the internal wall had also started to pull out. As the sheeting carried much of the tension from the uplift forces trans-ferred directly from the truss to the top plate, the fasteners that held the sheeting to the top and bottom plates of the framework carried sub-stantial load. The repeated nature of the loading caused those brads to work out of the pine in a similar manner to that described for the nails in the truss tie-down straps.

In spite of visible damage to the internal wall and one connection in a truss, the house, modified as indicated in Section 7, had endured the required number of cycles of 5/8 design load without major structural damage. The total vertical displacement at the eaves was approximately 20 mm and the horizontal displacement at the top of the wall was approx-imately 35 mm. The upward displacement was primarily due to the opening of joints and the horizontal displacement due to a combination of movement at joints, racking displacement of walls, and overturning of the complete house.

6.1.2 3/4 Design Load cycles

During this sequence, 2000 uplift load cycles were applied simultaneously with 200 lateral load cycles. Due to the large deflections registered by the house, few of the gauges recorded readings without significant error. However, the continuing demise of the house under the action of cyclic loading was well documented, with observations being made throughout the test.

The uplift deflections of 35 mm and lateral deflections of up to 60 mm were significantly larger than those expected from the previous tests indicating that non-linear behaviour of the house was dominating its performance. The non-linearity was primarily due to the overturning moments on the house being sufficient to lift the concrete footings on the windward

side of the house by 15 mm. This induced some out of plane bending into
the floor system. Throughout this series of loadings, a significant in-
crease in structural damage was observed.

Two batten straps had started to show signs of nails being worked out of
the truss top chords by the cyclic loading. The mechanism was similar to
that already described in section 6.1.1 with slip and grab on each cycle.

The trusses themselves continued to undergo further deterioration. Broken
nail plate teeth were found on the floor in the vicinity of two trusses
that later experienced complete joint failure in the Design Load cycles
sequence. While examinations did not reveal obvious damage to joints at
this stage, the presence of the broken teeth indicated that the failure
was progressive, and was taking place during the 3/4 Design Load sequence.

The demise of the internal wall also continued with the withdrawal of more
brads securing the hardboard sheeting and the complete failure of a stud
to bottom plate joint. On the windward side of the wide doorway, the damage
seemed to be mainly concentrated in the frame. The hardboard sheeting and
one stud had separated from the bottom plate of the wall, the sheeting was
still secured to the studs and top plate. Racking deflection was accommo-
dated by separation of the wall from the bottom plate, allowing the entire
wall to overturn. On the leeward side of the wide doorway, the hardboard
sheeting had separated from the frame almost entirely, and had allowed the
wall to deform from a rectangular shape to a parallelogram shape without
major damage to the timber wall framework.

However, while the damage to structural members was certainly very obvious
at this stage, the house continued to carry the applied load without large
scale failure.

6.1.3 Design Load cycles

After two uplift cycles at Design Load during which the concrete footing
blocks were lifted over 140 mm out of the ground on the loaded side of the
house, some failures occurred at framing anchors securing the joists to the
bearers.   The test was terminated at this point and the damage repaired.
The loads on those fasteners were well in excess of Design Load as no

account had been taken of internal pressure acting downwards on the floor.
The floor was therefore loaded with 200 litre drums filled with water to
apply a downwards load equivalent to the internal pressure. They were
distributed evenly over the floor area to give the same effect as a
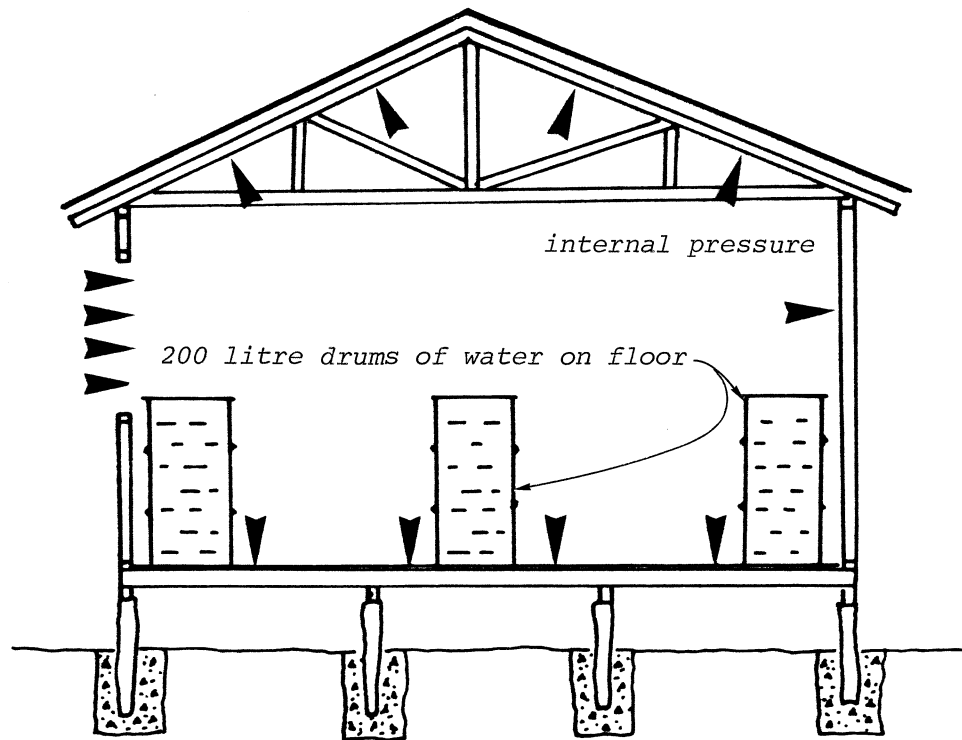uniformly distributed pressure loading as shown in Figure 13.

internal pressure

200 litre drums of water on floor

Figure 13   The Use of Drums of Water to Simulate
Internal Pressure

After 5 lateral load cycles and 50 uplift cycles at Design Load with com-
pensation for internal pressure, the concrete footing blocks were lifting
90 mm and the internal wall was totally destroyed as shown in Figure 14.
On the windward side of the large doorway, the wall had completely separated
from the bottom plate, and hence could no longer function as a bracing wall,
and on the leeward side, all of the hardboard cladding had worked loose
allowing the frame to deform in racking. In spite of the loss of this
wall as a restraining device, the house was still able to resist load.
Most of the deflection transducers were not capable of reading the large
range of deflections encountered on each cycle, so no further readings
were taken.

Also after 50 cycles of uplift at Design Load, it was noticed that the
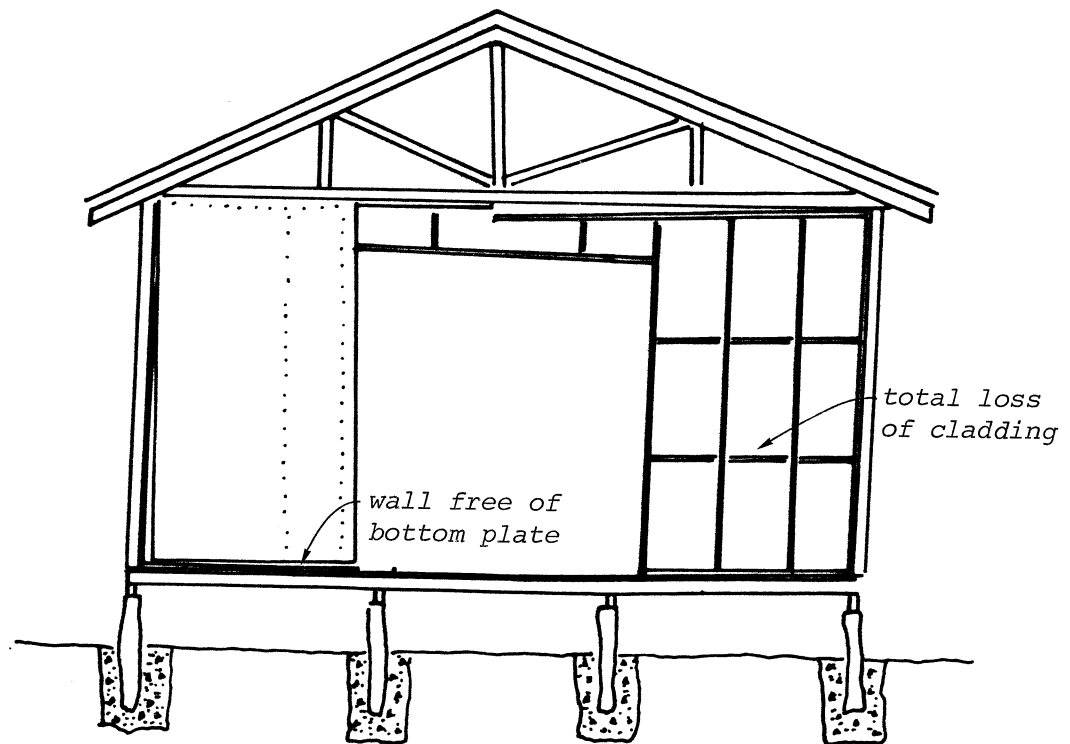apex joints on two trusses had failed due to fracture of the teeth in the

Figure 14   Failure of Internal Wall

nail plate. The failure mechanism was identical to that depicted in Figure 12,and as mentioned in Section 6.1.2 was probably initiated during the 3/4 design load cycles. However, inspite of the failure of this important joint, the house was still able to resist load. The tension that was previously carried through the apex of the truss by the failed plates was now being transmitted to the roof sheeting and over the apex by the ridge capping as shown in Figure 15. The ridge capping had deformed considerably, and the broken joint in the truss was opening at least 30 mm with each load cycle. Laboratory tests reported by Nash and Boughton (1981) show that the roofing nails at the ridge should be capable of transferring up to 16 kN of tension per truss over the apex before the ridge capping started to tear. An analysis of loads in the truss suggested that the tension at that point would have been less than 12 kN, so the good performance of the ridge capping was to be expected.

Applied Loads to Truss

flattened ridge capping

broken nails in timber

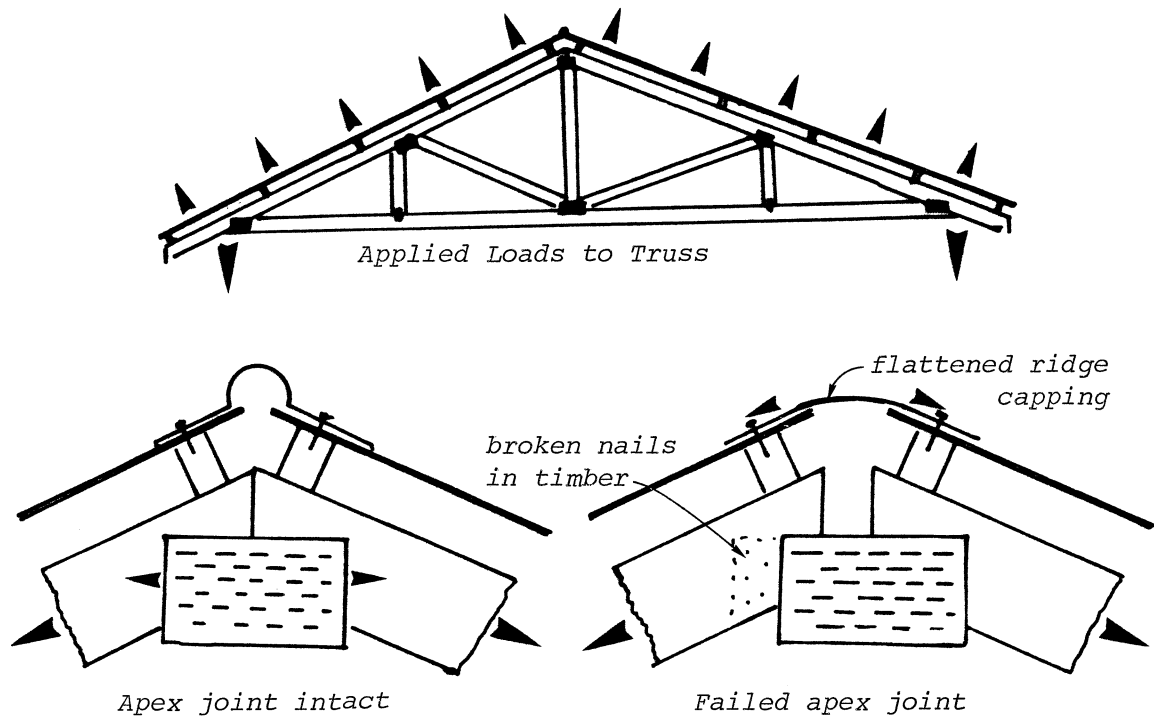*Apex joint intact*

Failed apex joint

Figure 15   Load Transfer through Ridge Capping

After the complete sequence of Design Loads, 200 uplift cycles and 20 lateral load cycles, the condition of the house was largely the same. During the Design Load cycles, an inadvertant uplift overload to over 130% design load gave rise to a significant   failure of the truss tie down system on the windward southern wall. The broken details were made good and the lateral loading equipment moved to now load the northern wall of the house.

6.2   Static Destructive Tests

Both lateral and uplift loads were applied to the house in increments of 10% of Design Load. Previous experiments suggested that failure could occur at between 120% and 130% of Design Load, so in the first instance

the simulated internal pressure acting downwards on the floor was left at the Design Load figure. On the application of 130% of Design Load, the house had pulled out of the ground in overturning so far that there was no travel left on the hydraulic rams to apply further loading. The appearance of the house is shown to scale in Figure 16.
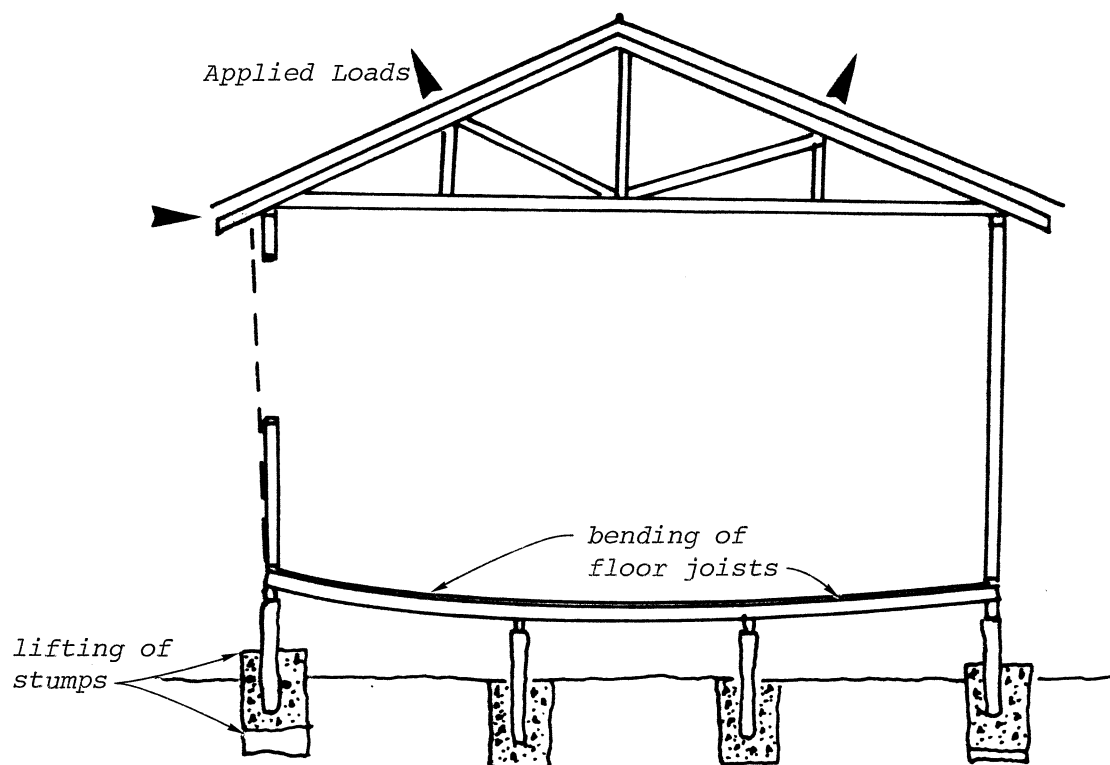
Figure 16   Overturning of the House - drawn to scale

The simulated internal pressure was increased to 140% of Design Load, and the test repeated with very little change in the house response. The concrete footing pads still pulled out of the ground far enough to warrant terminating the test at 130% Design Load. Other variations in the destructive testing were therefore sought to extract the maximum information from the tests.

6.2.1 Removal of an external wall panel

As a result of recent observations in Tonga, concern had been expressed with regard to the removal of wall panels to facilitate extension of the

houses (Eaton 1984). One complete 2.4 m x 2.4 m external wall panel was removed from the western end wall. This effectively halved the available length of bracing wall on that end of the building. Under these conditions, the fully instrumented house was again subjected to 10% increments of Design Load to 130% Design Load. The deflection of that wall of the house showed little change from the deflections prior to the removal of the wall panel. The most significant difference was an 80% increase in the movement of the windward wall relative to the leeward wall at that end. This was quite expected as load transfer from windward to leeward wall had previously been accomplished primarily by compression of the complete end wall. With one panel missing, the transfer had to be effected by the roof structure above, and this was a more flexible system. The windward wall deflection relative to the leeward wall was similar in magnitude to that relative deflection at other points in the house where the roof structure alone effected the transfer.

The overall apparent racking displacement of the end wall had increased by less than 10% indicating that the stiffness of the wall as a bracing member had changed very little. The apparent racking displacement was primarily due to overturning of the house rather than deformation of the wall in racking. When due account was taken of the overturning effect both the two panel wall on the eastern end and the single panel wall on the western end behaved as effectively rigid bracing members. Thus, the removal of one end wall panel made no significant difference to the force distribution within the house, and the overall strength of the house was still sufficient to resist 130% Design Load without additional damage.

After the release of the load, approximately 10 mm lateral displacement at the top of the windward wall remained, preventing the easy replacement of the removed panel. The windward wall was restored to its correct position using hydraulic jacks and the panel was replaced in the western wall in readiness for the next test.

6.2.2 Load to destruction

The drums of water inside the house, that had simulated the internal pressure effect on the floor, were moved to position most of them immediately adjacent to the windward wall. This increased the resistance to overturning. Under

these conditions, 130% Design Load could be sustained with less distress
to the footings. However, at marginally above 130% Design Load significant
failure of the house occurred.

The primary cause of failure was a brittle fracture of a truss top chord.
The line of the break ran through defect free timber immediately adjacent
to the overbatten tie down, and was almost exactly perpendicular to the
grain. This suggested that shear stresses may have played a significant
role in the fracture. Calculations showed that at the failure load, the
shear stress in the top chord was 3.3 MPa, over 2.2 times the basic
allowable working shear stress at joints designated in AS 1720, the SAA
Timber Engineering Code. Also the tension stress at failure was calculated
at 7.3 MPa or over 1.2 times the basic allowable working tensile stress
for wind loads. Failure was clearly due to the combination of these two
stresses being in excess of allowable.

A sketch of the failure is given in Figure 17. It shows the failure of
the top chord, nearly perpendicular to the grain, and the other associated
failures in the vicinity of the primary failure. A possible sequence of
failure is as follows: as the break occurred, the major part of the upper
chord would have been lifted clear of the break by the uplift forces. The
roof sheeting would have had to bend over the discontinuity, increasing
the load on the lowest batten which was still attached to the separated
part of the top chord near the heel of the truss. It is postulated that
this overload caused the failure of the batten fixing straps at that
point by nail withdrawal. The loss of this strap placed a significant
bending moment on the bottom chord of the truss. This timber member was
the sole remaining connection between the two parts of the broken truss
and its failure was a classical bending fracture.

The combination of these three failures, shown in Figure 17, meant that
broken truss over the internal partition was not transmitting uplift
loads to the walls at the southern end. The battens therefore carried
this load to adjacent trusses causing overloading of their truss tie
down. Under these circumstances, the actually applied load at the truss
ties could have been up to 2 times design load. These details failed,
causing overloading of all of the other truss restraints, effectively
freeing the roof structure from the southern or leeward wall.

Clean break of
top chord in shear
and tension

Broken nail plate teeth
see Figure 15

broken nail plate teeth
see Figure 10

flexural failure of
bottom chord

elongated
holes

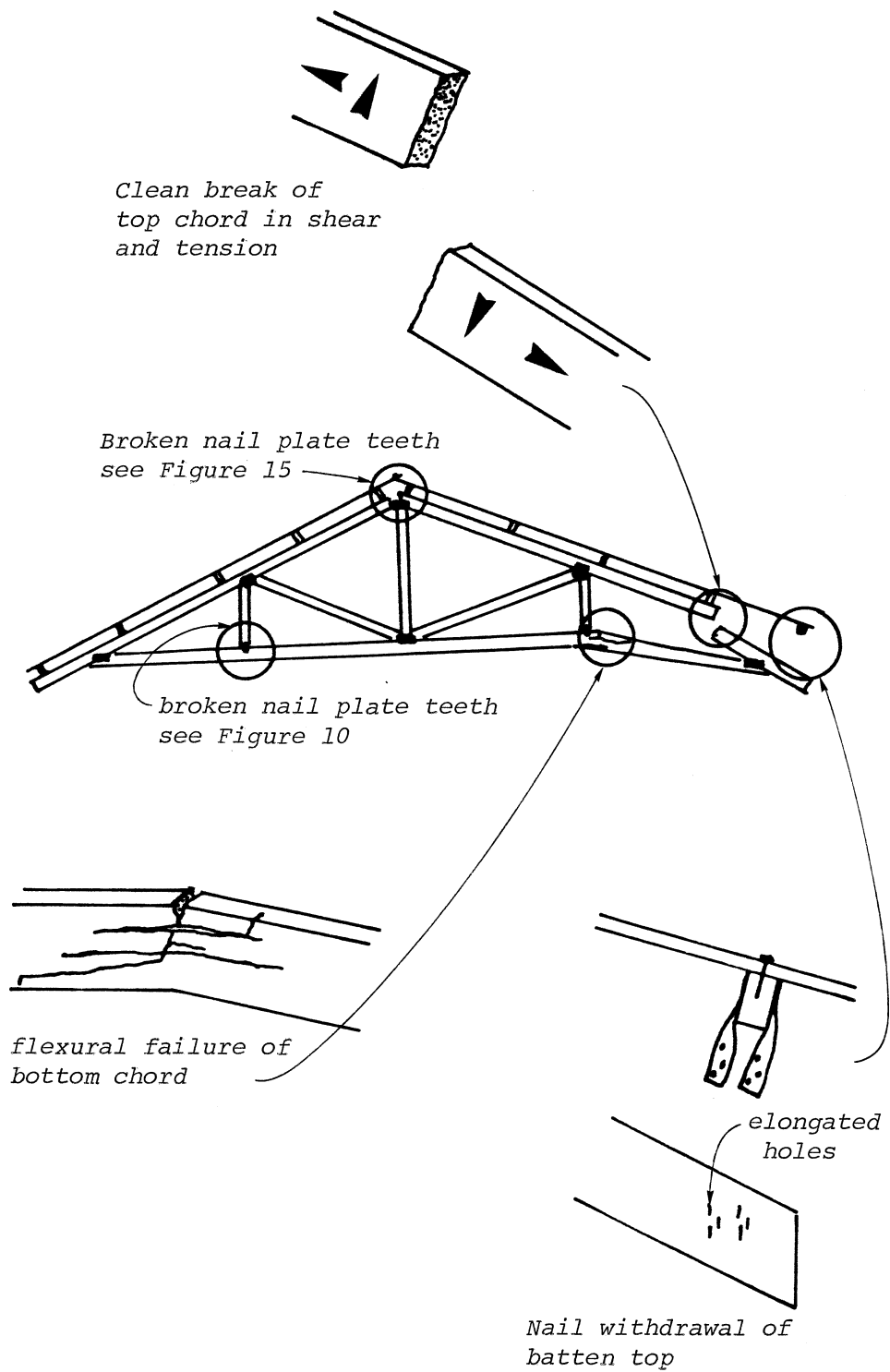Nail withdrawal of
batten top

Figure 17   Details of Truss Failures

The roofing system had been performing a vital structural role, in that it braced the top of the windward wall using diaphragm action. The roof structure, now freed from one complete wall of the house was less effective in performing that task and shed some of its load to the windward wall top plate. The top plate failed in two places due to a combination of bending and shear stresses. The plan of the house given in Figure 18 shows the location of all of the failures associated with this destructive test.

Failure of top chord

Apex joint failure *Apex joint failure* *Apex joint failure*
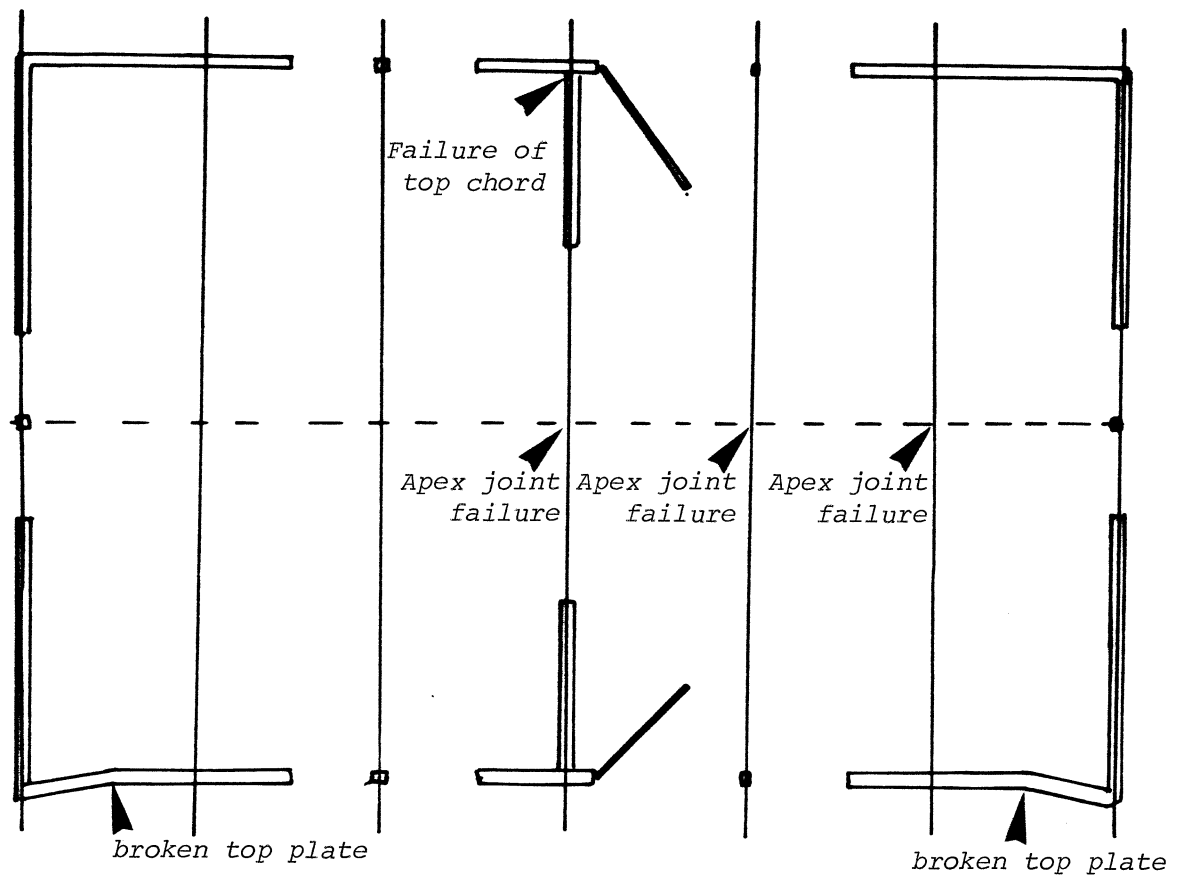
broken top plate *broken top plate*

Figure 18   Plan showing Locations of Detail Failures

The study of the roof sheeting as a diaphragm produced some very interesting results. The roof had been fastened in such a manner that most lap joints had only 2 nails common to both sheets, these being the nails at the ridge capping and at the eaves. Theoretically, such a lap should have only been capable of transmitting 2.8 kN in shear between the two sheets. However in the course of the tests described in this section, approximately 15 kN was carried across some lap joints. In these cases the lap had slipped, by

up to 15 mm, and it is postulated that this movement had mobilised the resistance of nails near to the lap and transferred the greater part of the shear force to the timber battens.

The diaphragm action of the roof was much stronger than anticipated due to a sharing of load between roofing and timber, once sufficient sheet tearing at the lap had occurred to mobilise the shear transfer. Nash and Boughton (1981) give loads at which the tearing process commences. These are 0.25 times design load based on two nails per lap joint and 2.8 times Design Load assuming full load transfer to the battens. Clearly, the results of these tests indicate that the actual mechanism of load transfer across lap joints lies between the two extremes.

6.3 Significance of the Failure Loads

Each of the failures observed during the course of the tests will be discussed in terms of its relationship to design load, and likelihood in a real cyclone event. Table 2 summarises this information.

Table 2

SIGNIFICANCE OF FAILURE LOADS

Failure	Load Factors with respect to	
	Design Load	Wind loads from AS 1170 Part 2 - (1983) or Wind Tunnel Study.
Building overturning	1.3	2.4 (Wind Tunnel Study)
Truss Tie down straps - open country	0.6	0.8 (AS 1170)
- protected terrain	1.5	2.0 (AS 1170)
overbatten - open country	> 1.3	> 1.7 (AS 1170)
truss	1.3	1.2 (AS 1170)

6.3.1 Lifting of footings from the ground

The calculated resistance to withdrawal of the footings from the ground

provided by the weight of the house above floor level and the mass of the footings was 28 kN per side of the house. Some lifting of the footings was first observed when the applied loads produced 36 kN uplift on the windward wall. This was a combination of uplift at roof level and overturning due to the applied lateral load. Thus the applied load was approximately 130% of the nominal resistance. The extra resistance was provided by bending of the floor on withdrawal of the footings under the windward wall, transferring some uplift to other footings near the centre of the house as shown in Figure 16.

Calculations show that had the uplift and moment combined to reach 75 kN uplift at the windward wall, uncontrolled overturning would have resulted. 1.3 times Design Load, which was the point at which the uncontrolled overturning was observed, produced 77 kN of uplift at the windward wall.

As observed in Section 4.4, the actual Design Loads do not truly reflect the loads experienced on the footings during a tropical cyclone. The total uplift on the footings and overturning moment at ground level as evaluated in the wind tunnel study combined to give an expected uplift at the base of the windward wall of 32 kN. This is marginally greater than the calculated resistance of the house with no footing movement at all (28 kN), but much less than the ultimate load achieved for overturning. The ultimate load for overturning was 2.4 times the expected overturning load based on results of the wind tunnel study.

Thus although the house showed a pronounced tendency to roll out of the ground at 1.3 times the Design Load, and significant withdrawal of footings from the ground at Design Load, the above analysis shows that little movement of the footings is expected during a design tropical cyclone, and that the load factor against complete overturning was 2.4. The authors do not recommend any changes to footing details, as the wind tunnel study of total wind loads gave loads that indicated the true footing loads would have been significantly less than the capacity of the house to resist them.

6.3.2 Failure of the twisted straps securing trusses

The systematic failure of these straps occurred at 5/8 Design Load, and would have led to significant failure of the whole house, had not

loading been terminated. The first strap to fail did so after 2500 uplift cycles with a total roof uplift of 64 kN per cycle. As noted in Table 1 in Section 4.4, the use of the Australian Wind Loading Code, AS 1170 Part 2 - 1983 results in lower uplift loads being obtained. Even so, the 64 kN per cycle is approximately 3/4 of the wind load derived from AS 1170 Part 2 - 1983. Thus had the cycles been based on wind loads calculated using AS 1170 Part 2 - 1983, it is quite probable that the strap would have failed during the 2000 cycles at 3/4 wind load. On this basis, the authors make recommendations regarding the use of these straps in Section 8.

At lower wind speeds, the strap performance may have proved adequate. For example, under sheltered suburban conditions - Australian Standard terrain category 3 - the calculated uplift load based on AS 1170 Part 2 - 1983 pressure coefficients reduced to 30 kN. Based on that wind load, the straps had been subjected to 2.1 times design load for 2500 cycles. They probably would have survived the prescribed sequence of load cycling at lower loads and still had reserves of strength to spare, according to classical exponential metal fatigue curves.

The replacement detail - an overbatten fastening system detailed in Section 7 - endured the Design Load based cyclic loading sequence without permanent damage, and successfully sustained a combined load equivalent to 1.3 times Design Load. Using the wind loads derived from AS 1170 Part 2 - 1983, the load factor achieved is in excess of 1.7.

6.3.3 Failure of metal nail plates in trusses

At least five metal nail plates in trusses had failed by the completion of the cyclic and overload sequence of tests, and yet none of these contributed to the final catastrophic failure of the house. Thus although the plates did not have sufficient strength to carry the Design Loads in conjunction with cyclic loading at lower loads, their loss could be accommodated by other elements of the house assuming their structural role. The most notable case was the ridge capping which could transmit tensions over the apex of the truss at 1.3 times applied Design Load with further reserves of strength as yet untapped.

The failure of the nail plates, although presenting a maintenance problem after the passage of the cyclone does not pose a crucial threat to the structural integrity of the house during the cyclone.

6.3.4 Failure of the internal wall

While the failure of the internal partition did prove spectacular, and would require maintenance after the passage of a design cyclone, the other structural mechanisms within the house could pick up and redistribute the load shed by the wall upon failure. The roof sheeting diaphragms could effectively transfer the lateral load away from the windward wall to the end walls and, as indicated in Section 6.2, had reserves of strength even at 1.3 times Design Load. Likewise the end walls were showing no sign of distress at the house ultimate load and could have sustained significantly higher bracing loads.

The internal partion wall was a largely redundant structural feature, and its loss was compensated for by load transfer to other bracing elements.

6.3.5 Failure of truss top chord

This failure was the crucial one that gave the ultimate load for the whole house. Again, the load factor based on Design Loads was 1.3. However, using the recently published Australian Standard AS 1170 Part 2 - 1983 to calculate the design loads increased the load factor for the trusses to over 1.7 for wind normal to the long axis of the house. For wind normal to the end wall, a higher load is applied to the trusses and the load factor reduces to 1.2. Under these conditions no lateral load is transferred by the truss tie down details and therefore the total load in the truss tie down is less than that for the wind normal to the long axis.

7.   REDESIGN OF HOLD-DOWN DETAIL

Failure of the twisted metal strap detail used to secure the trusses to studs has been outlined in Section 6.

Because the test loading system was based on a fail-safe technique, the roof structure had been lifted only a few centimetres off the wall when failure of the straps occurred. The house, although deemed to have structurally failed, was quite sound except for a few broken metal straps. Thus, having established a weakness in the original design, the authors siezed the opportunity to implement repairs so that the test programme could continue and therefore recommendations for improving the system could be made to the Tongan Government.
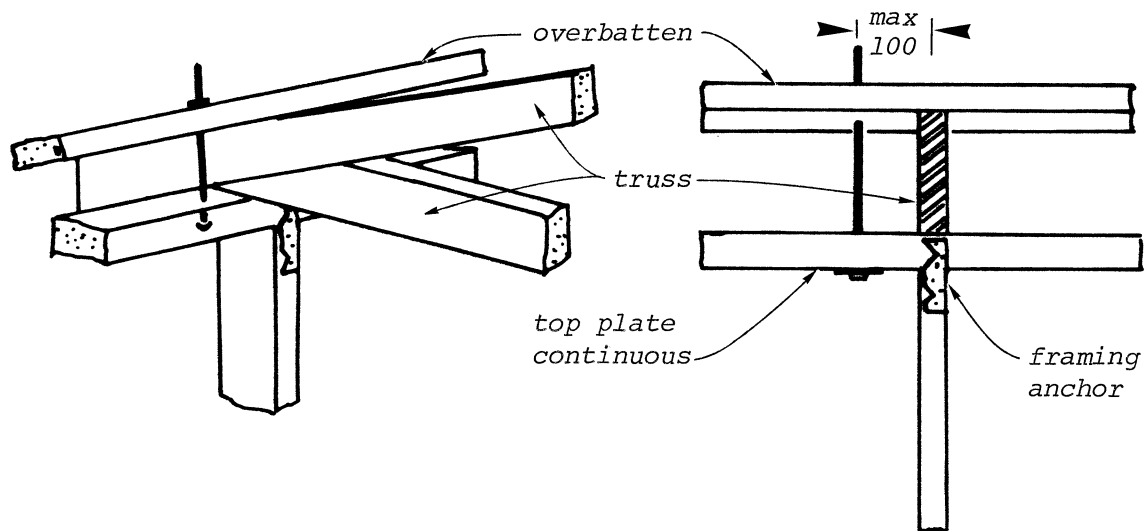
Figure 19   Bolting of Overbatten

One constraint was put on the repairs. They had to be simple and easy to effect in the existing houses in Tonga. An obvious solution was the use of overbattens, that is lengths of timber that could be installed on top of the top chord of the trusses directly over the external walls. These overbattens were bolted to the top plate of the wall, adjacent to the trusses. Figure 19 illustrates the system.

As calculations showed that the strength of the system was somewhat marginal, and the distribution of forces difficult to predict, some sample specimens were tested in the laboratory.

7.1   Laboratory Testing

In the original Tongan hurricane house roof trusses were purposely located immediately over wall studs, and hold-down straps secured the trusses directly to the studs. The concept of this was good engineering design as the forces bypassed the top plate which was secured somewhat eccentrically. However the proposed overbatten design is reliant upon putting forces into the top plate and taking them out via the plywood and the framing anchor at the stud/plate joint.

As previously mentioned, calculations based on static loading indicated that the joint may be satisfactory. But this did not take account of uneven loading on either the plywood or the framing anchor, nor the effect of cyclic loading. Therefore some laboratory test specimens were fabricated to simulate the joint. Figure 20 shows a typical specimen.

For the first test, the two plate members were gripped in a testing machine and slowly pulled apart. The specimen failed at a load of 8.5 kN, but the fracture occurred at a knot in one of the plate members.

Two other specimens were both tested in cyclic loading. They resisted 8000 cycles at 8.5 kN with only a small amount of cracking of the timber plates. Both specimens failed during cycling to 10 kN, the first after 250 cycles and the second after 1250 cycles. These failing loads compare with the design uplift reaction of a truss calculated as approximately 8.2 kN.
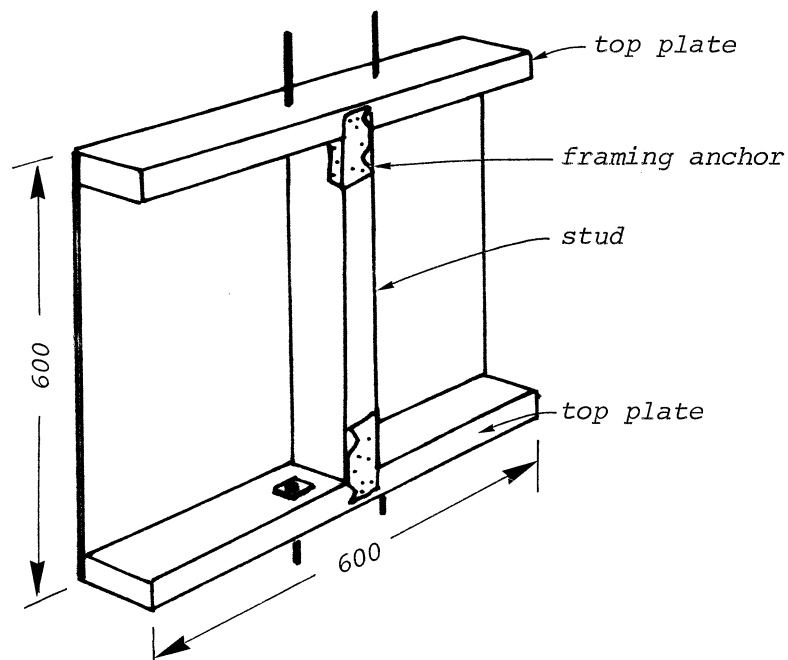
Figure 20 - Laboratory Tests Specimen - Top
Plate to Stud Joint

Although the failure load of the laboratory specimens was little in excess
of the design uplift load, a few other aspects should be taken into con-
sideration. Because each specimen consisted of two prototype joints, one
at the top plate and the other at the bottom plate, the failing load must
represent the lower of two possible values. Thus the tests yielded the
lower two of four joint strengths.

A further consideration relates to the length of the test specimen compared
with the spacing of the trusses. In making the test specimen 600 mm long
with the stud/plate joint in the middle, the assumption was made that most
of the applied uplift force in the house would be transferred straight
into the stud and none of it would to transferred more than 300 mm sideways.
However if some of the force is transferred further sideways than assumed,
the laboratory results are conservative.

Considering the combination of these conservative effects the authors
decided to use the overbatten system of securing the roof trusses, instead
of the light gauge strapping system. Overbattens were therefore installed
in the test house. At those trusses located over a joint in the wall panel,

two M12 bolts were used,one each side of the truss. At the other trusses only one bolt was installed. Whilst one bolt is sufficient to transfer the uplift forces to the wall, the discontinuity of the top plate meant that the framing anchor securing it to the stud was positioned at the extreme end of the plate. It is probable that the uplift force would cause the plate to split and not allow the framing anchor to develop its full strength. Hence two bolts were used for those situations. Figure 21 illustrates this case.

Testing of the house was continued using this system.
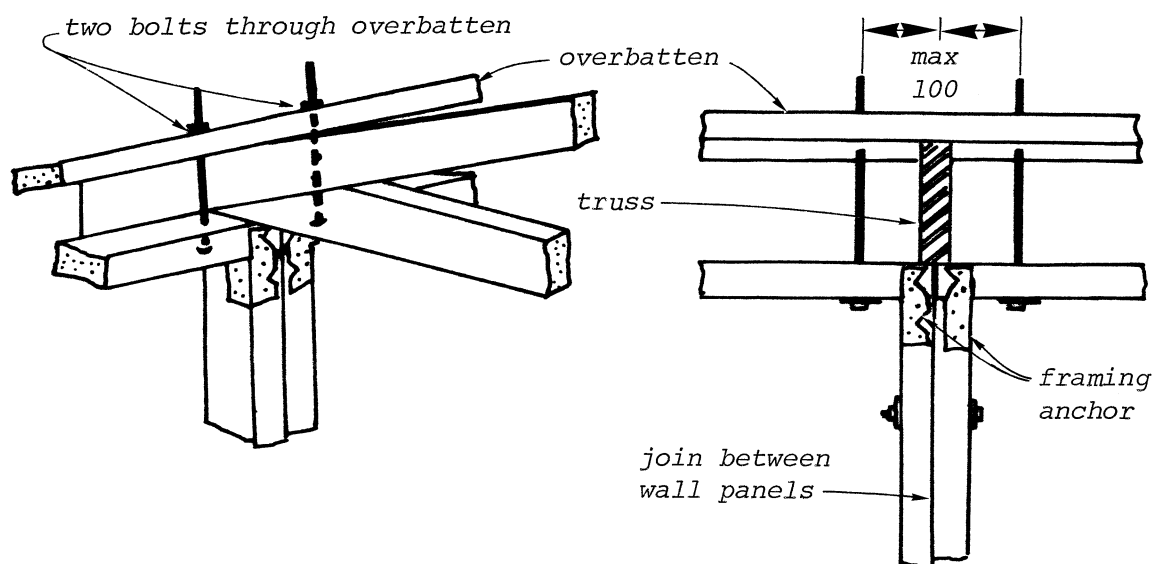

Figure 21   Overbatten at Joints in Top Plate

7.2   Fitting Overbattens to Existing Houses

An overbatten system is easy to install in an unlined gable ended cottage, such as those already built in Tonga. The following points are offered

as a simple set of instructions by which the overbattens can be installed.

1. Select 2 or 3 suitable lengths of 70 x 45 mm seasoned pine, or
    equivalent.

2. Join the pieces together with toothed plate connectors to form a
    single length greater than the length of the house.

3. With the overbatten on its flat, mark the relative position of
    each truss from one end and if necessary adjust the layout so
    that there is no serious defect within 100 mm of a truss position.
    An ideal layout would be to have any joint in the overbatten
    located directly over a truss at the joint between wall panels.
    Thus there would be a bolt on either side of the joint.

4. Cut the overbatten to length, lift the barge board at one end of
    the roof and slide in the overbatten on its flat, directly over
    one wall.

5. Position the overbatten so that the truss marks align with the
    trusses. Drill the top plate and overbatten with 12 mm clearance
    holes, as near to the truss as practicable, but no more than 100
    mm away.

6. Insert 225 mm long M12 bolts with a 50 x 50 mm square washer at
    each end.

7. Tighten bolts, replace barge board and repeat the procedure for
    the overbatten at the other end of the trusses.

8. RECOMMENDATIONS

8.1 The Modified Hurricane House

The brief given to the Cyclone Testing Station was to determine the strength
of the Tongan hurricane house when subjected to simulated cyclone wind
forces likely to occur at exposed locations. In this context exposed loca-
tions have been taken to mean terrain category 1 as defined in Section 4.

Thus houses facing the sea or a large lagoon can be considered to be in
exposed locations. Nearby houses may experience wind forces of similar
magnitude.

As the test programme highlighted a weakness in the original design in
respect of light gauge steel straps, it is recommended that they are not
used in exposed locations.

It is recommended that existing houses in exposed locations be upgraded
by the installation of overbattens as described, and that new houses in
such locations include the overbatten system as a matter of course. This
should give them a strength similar to the modified test house.

It could be considered that the performance of the trusses in the hurricane
house is a cause for concern. The consequences of having a truss fail
at 1.3 times its Design Load (or 1.2 times the Wind Load for wind direction
parallel to the ridge) must be assessed. This margin of strength over the
Design Load allows little room for variability between the strength of
trusses in the test house and the strength of trusses in houses built in
Tonga. Variability in strength can arise by using different species or
grades of timber, by using different size truss plates, by variations in
fabrication or by incorrect handling during erection.

In considering the ramifications of the test results, due regard must be
given to the fact that economic factors in developing countries may well
dictate lower safety margins. Also, the task of improving the strength
of all roof trusses in existing houses in exposed areas would be considerable.

The final decision about whether or not to upgrade the roof trusses can
only be made by the MOW who would assess the cost involved compared with
the risk of occurrence.

8.2   The Original Hurricane House

Although the hurricane house was designed for use in exposed locations,
some will probably be built in villages and inland areas where they will
gain some shielding by surrounding objects. In such sheltered locations
the original house construction, including the light gauge steel straps
may be satisfactory.

Sheltered locations can be considered as terrain category 3 from the
Australian code. This category has been defined in Section 4. The design
wind forces on a house in such a sheltered location are only about 40%
of those on a house in exposed terrain. Thus the value of 5/8 Design
Load for exposed locations (which caused failure of the strap after about
4000 cycles) is equivalent to about 2.0 times the wind load for sheltered
areas, calculated according to AS 1170 Part 2 - 1983. Sheltered houses
would not have to resist cyclic loading to this degree and therefore
the strapped system may be satisfactory.

It is recommended that houses in such sheltered locations need not be
upgraded, provided there is no risk of the steel straps or their fasten-
ings deteriorating due to rust or some other means.
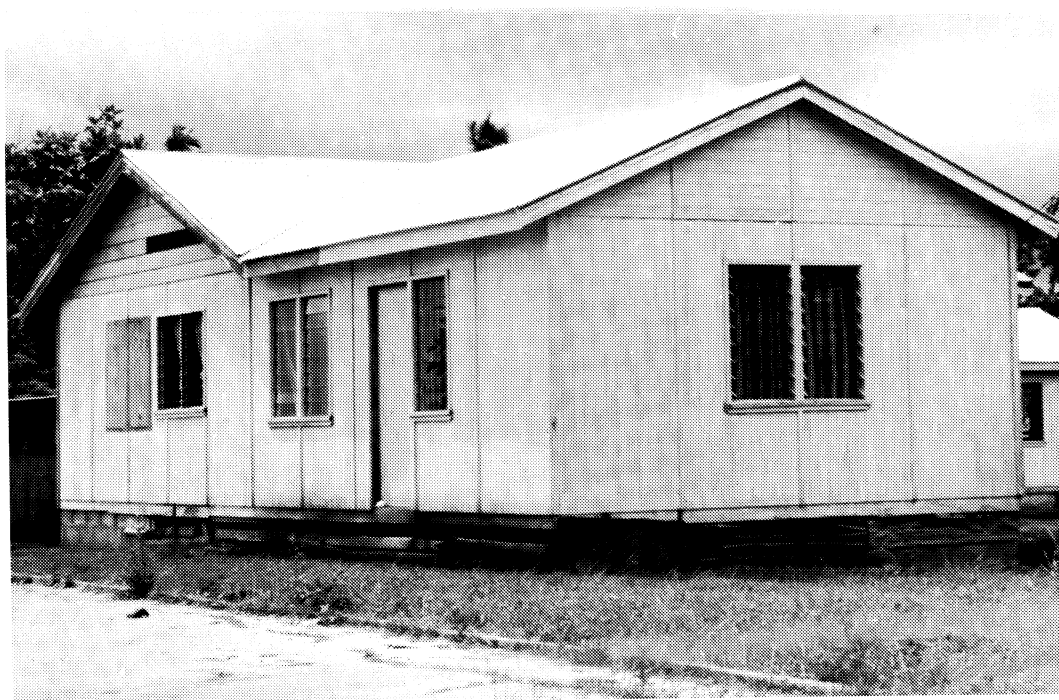

Figure 22   L-shaped extended house.

8.3   Additions to the Hurricane House

8.3.1   In the longitudinal direction

In some instances the basic 35 m² house has been found to be too small

and additions have been made. Figure 22 shows how a house has been
extended to an L-shaped plan. This form of extension is probably the
easiest to achieve as it involves removal of one or two panels in the
so-called non-loadbearing end wall. (Whilst the end wall may not
support the roof structure it certainly acts in a load bearing capacity
by bracing the house against lateral wind forces!)

As the test on the house with one end panel removed showed that there
was still significant racking strength available, it appears that
extending the house in this manner would not severely weaken the structure.
No tests were conducted with both panels removed from the end wall, but
even that situation could be satisfactory, provided that the structural
strength of the extension was similar to that of the original house.

It is therefore recommended that L-shaped extensions or extensions in the
longitudinal direction be permitted provided that they be made to the same
structural standards as the original house. The extensions should
preferrably be constructed from panels and trusses supplied by the MOW.

8.3.2 In the lateral direction

Figure 23 shows extensive additions to the basic hurricane house. Extra
rooms have been added to both front and back to give a floor area of
about 2½ times the original.

Extreme care must be taken with any extensions in the lateral direction,
such as shown in Figure 23. If the roof structure of the extension is
supported by the longitudinal load-bearing wall of the original house,
that wall may be severely overloaded during a cyclone. Even worse, if
the lean-to roof structure is fastened to the overhang of the trusses,
the truss hold-down detail will be severely overloaded. The situation
would be exacerbated if any of the studs were removed to provide a wider
door opening or to gain access to the bedroom.

It should be remembered that laboratory tests on the wall joint (see
Section 7.1) showed it to have only a marginal reserve of strength.
This reserve would almost certainly be eroded by the additional forces
due to wind uplift on the roof of the lean-to.

Figure 23   Additions to Tongan house.

No specific recommendations for this type of addition can be made. Each
case should be assessed separately. In general the strength of the wall,
and possibly the truss hold-down connections, should be improved. The
addition of plywood lining on the internal face of the wall, fastened
in the same manner as the external plywood, would certainly increase
the strength of the wall. The use of two bolts per truss as illustrated
in Figure 21 for all trusses would help by reducing stresses in both
the top plate and the overbatten.

The overbatten system should be used for both sheltered and exposed
locations, for houses incorporating extensions.

8.4   The Internal Wall

While the early demise of the internal wall did not significantly affect the overall strength of the house, its destruction during the course of the cyclone could result in a terrifying experience for the occupants. Its early loss may lead to the misjudgement of the strength of the house by those sheltering in it and their premature departure.

The authors recommend that no action be taken with regard to changing existing houses, as the cost involved in any upgrading will not lead to any increase in the overall performance of the building. However for future houses, the MOW may consider alternative details for the internal wall in order to alleviate any fears its poor performance in a cyclone may create. Two suggests for alternatives are offered:

(i)   As the wall serves no structural purpose, it could be completely removed and replaced with a curtain suspended from the bottom chord of the truss. In this way, the performance of the truss would be similar to the other unsupported trusses in the house.

(ii)   If a lined wall is required for functional reasons it should be stiffened and strengthened so that it can carry the uplift loads to the floor. The hardboard lining may still prove strong enough but should be fixed with 2.8 mm flat headed nails at 100 mm spacing around the perimeter and 200 mm spacing on the internal studs. Alternatively plywood sheeting similar to that used on external walls would certainly prove adequate to carry the load.

The roof truss should also be lined so that its stiffness matches that of the internal wall. If the truss is left unlined premature failures of nail plates will occur. The truss lining should be fixed in the manner specified for the internal wall.

Lining of both the wall and truss can be placed on one side only.

8.5   The Pacific Area Hurricane House

A considerable amount of effort has been directed towards the development of the Tongan hurricane house. First was the design of a cyclone-resistant structure and the development of a simple prefabricated system to allow factory production. The second phase was the implementation of the concept by establishing factories to construct the panels and trusses within strict tolerances. The third stage was the arrangement of suitable transport by land and by sea to deliver the components, and finally the organization of skilled teams to erect the components to form a cyclone resistant house.

The Tongan Government should be congratulated for this excellent achievement.

Although it may have been somewhat of an afterthought, the culmination of the hurricane house project was the simulated cyclone wind testing programme described in this report. These tests verified the concept as being cyclone resistant with minor modifications.

It is the authors' opinion that the hurricane house project should not fall into obeyance on completion of the target number of houses to replace those destroyed. In fact it should be continued for future construction on Tonga and also should be exported to other Pacific island countries in the tropical cyclone zone. There should be a market for cyclone resistant construction in countries such as Samoa, Niue, Solomon Islands and even Fiji. Maybe the architecture would need to be changed to satisfy local traditions, but the basic method of prefabricating strong building components in a factory and assembling them on site, under supervision, should be retained.

It is therefore recommended that the Tongan hurricane house concept be used as the basis for a Pacific Area hurricane house. Development of the scheme should begin immediately and construction of the houses should commence before the next serious cyclone hits the area, not after. The Cyclone Testing Station and the Building Research Establishment are planning further action to this end.

9.   CONCLUSIONS

In a simple structure such as the Tongan hurricane house, with no internal
lining or ceiling and few walls, there appears to be little avenue for the
sharing of load between structural elements as was very evident in tests
on other houses. However despite this, the Tongan house managed to support
significant loads while a number of truss plates were broken. The most
obvious example of this load sharing was at the apex joint of some trusses,
where the ridge capping carried load instead of the broken truss nail plate.

This mechanism for survival by the house would never be anticipated from
laboratory tests on individual walls or roof trusses, and therefore it
emphasizes the benefits to be gained from testing complete houses even
if they are simple structures such as this one.

The following conclusions can be drawn from the tests on the Tongan hurricane
house.

(a) The house in its original condition had its first failure after
     about 2500 cycles of 5/8 Design Load in uplift and 250 cycles of
     5/8 Design Racking Load when a light gauge metal strap securing a
     truss to the wall broke in fatigue loading. The loading was
     equivalent to 0.8 times an alternative wind load calculated from
     the provisions of the latest Australian Wind Loading Code.

(b) Other metal straps failed after about 4100 cycles of the same
     loading, causing the whole house to be deemed to have failed.

(c) The modified house, with its overbattens, resisted the full
     complement of cyclic loading although some truss plate connectors
     fractured.

(d) The modified house failed at 1.3 times Design Uplift and Racking
     Loads when the the chord of a truss broke. This loading was
     equivalent to 1.7 times the alternative wind load calculated from
     the provisions of the latest Australian Wind Loading Code, for
     wind acting normal to the length of the house. Conversely this
     factor would reduce to 1.2 for calculations based on wind acting
     parallel to the length of the house.

(e) Although the test house deflected significantly due to both
     overturning and lateral bending of the windward wall, the
     actual racking stiffness of the house was quite high.

(f) The roof sheeting performed adequately as a bracing diaphragm in
     transferring the lateral wind forces to the end walls, but the
     detail attaching trusses to walls was such that it allowed signifi-
     cant lateral bending of the windward wall.

(g) The internal partition wall acted as a structural element although
     it had not been designed as such. Because the central roof truss
     was nailed to that wall at its top plate, the wall resisted some
     uplift forces as well as racking forces before it failed.

(h) Removal of one of the two panels from an end wall made no apparent
     difference to the strength of the house, but the windward wall
     deflected slightly more under these conditions.

(i) Wind tunnel tests indicate that although the footings started to
     pull out of the ground during test this is most unlikely to occur
     in practice.

(j) The modified house satisifed the brief outlined by the Tongan
     Government, that it withstand the simulated wind forces with
     some reserve in strength. In so doing it may form the basis
     of a Pacific Area hurricane house.

(k) The results given in this report apply only to houses built
     exactly the same as the test house. Houses built with different
     components or cladding materials, or those extended or modified
     in any way, may have a strength and stiffness different from the
     test house.

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