CYCLONE TESTING STATION

SIMULATED CYCLONE WIND LOADING OF A NU-STEEL HOUSE

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SYNOPSIS

Simulated cyclone wind loading tests were conducted on a steel framed house supplied by Nu-Steel and erected by a specialist fabricator. The house had been designed for terrain category 3 (suburban) exposure in a tropical cyclone prone area. Large steel frames were used to apply the simulated wind loading. The response of the house was measured by electronic displacement gauges and fed into a portable computer used to store data and to control the cyclic loading programme.

Preliminary racking tests were conducted during construction to ascertain the change in lateral response by adding the roofing and ceiling. They showed that the lining elements acted as stiff diaphragms. Combined cyclic uplift and lateral loading was applied to the house to simulate the gustiness that occurs during a tropical cyclone. During the latter part of this cyclic loading a hold-down bolt started to withdraw from the slab, but the house still resisted the prescribed loading without fracturing.

During the overload phase of the test for wind parallel to the ridge a number of batten/rafter joints broke. A re-assessment of the design loads showed that the load at failure was just in excess of the target load, but it was still below the load required to resist high local uplift pressures.

The house was very stiff and strong under lateral forces. The racking response was unaffected by the removal of the internal diagonal braces. Despite the effective removal of three internal walls the house still resisted 2.7 times lateral design load and had a relatively small lateral deflection.

In the final test a lintel beam was loaded in combined uplift and lateral loading to twice design load without any obvious failure.

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1. INTRODUCTION

This steel framed house is the last in a series of three research houses to be built and tested for the Australian Uniform Building Regulations Co-ordinating Council. The first was a timber framed brick veneer house built for cyclone prone areas of Australia. The second test house was also of timber framed brick veneer construction, but it was designed and built for non-cyclone regions. It was referred to as the Melbourne style house. The performance of those houses has been reported in the Station's Technical Report series (Reardon, 1986; Reardon & Mahendran, 1988).

In the Cyclone Testing Station's overall research programme into the performance of housing under simulated wind loading, this Nu-Steel house is the sixth new house to be tested. The performance of the other houses has also been published in the Station's Technical Report series.

Steel framed house construction was chosen for the test programme because it represents a relatively new form of construction that has different advantages and problems from traditional timber framed housing. Also this form of construction is reportedly gaining an increasing proportion of the cottage building market.

The Nu-Steel house is typical of steel framed house construction which has kept the traditional wall and roof framing members. The system does not use prefabricated modular wall panels, such as were used for the Logan Unit test house (Reardon & Boughton, 1984). However because it was designed as a kit house for the owner/builder, the Nu-Steel system has constraints in its dimensions. The standard width is 7.5 metres and the length is a multiple of 1.2 m, which is also the truss spacing. Having a standard width of house would facilitate the extrapolation of the results of the tests on this house to other houses.

Because of the possibility of fatigue of the light gauge steel members under cyclic loading, it was considered appropriate to test a house designed to withstand tropical cyclone wind forces. The test house had been designed for terrain category 3 (suburban) areas.

The main reason for testing full size houses rather than components is to investigate the effects of load sharing of the various elements. Although it is much less costly to test individual components such as bracing walls and the like in isolation, such testing ignores any contribution to strength that may be supplied by adjacent stronger or stiffer elements. This can occur in a number of different ways. The most obvious is the tendency for some supposedly non-structural elements to act in a structural manner. For example the internal lining of a house is generally not designed to contribute to the structural strength, yet the previous tests on houses have shown that this lining plays the predominant role in bracing the structure against lateral wind pressures. In a similar manner, the roofing can

provide lateral bracing. Another manner in which load sharing can occur is when a stiffer member gives some support to a more flexible one. In so doing it effectively attracts some of the applied load away from the the more flexible member.

2. THE TEST HOUSE

2.1 Design

The house chosen by Nu-Steel for test was their "Matilda" plan. It is a four bedroom house 15.6 m long and 7.5 m wide. Figures 1 & 2 show elevations and a plan. The gable ended roof had a 20° pitch and was clad with Lysaght's Trimdek steel roof sheeting. The house was clad externally with James Hardie Industries' Hardiplank wall cladding. The internal lining was plasterboard on most walls and the ceiling, but fibre cement board was used on the walls of the bath room. In the kitchen moisture resistant plasterboard was used as the wall lining.

This house was designed in 1984 and its wind resistance would therefore have been in accordance the 1983 edition of the Wind Loading code. From this code a design wind speed of 42 metres per second can be calculated for terrain category 3 of a cyclone prone area. The Cyclone Testing Station did not consider it important to check the design calculations for the house, as the test programme was meant to do that.

The test house comprised only those components which were considered to contribute to its structural strength. Therefore there was no plumbing, electric wiring, kitchen benches or any other such element that would obviously contribute nothing to the structural strength. The built-in cupboards and wardrobes that were supplied as part of the kit were installed, because some of them may have effected the strength if they were left out. No doors or windows were installed in the house, but during loading provision was made to take account of the effects of wind acting on them.

2.2 Construction

The Nu-Steel housing system has been developed around the owner/builder market. Virtually all of the components are prefabricated in a factory and transported to the building site. Each wall is made to the appropriate length and its location is marked on it. The roof trusses are also prefabricated. Construction of the house then becomes an accurate assembly of the components, rather than the skilful cutting and fitting of components.

The manufacturer supplies a comprehensive manual outlining the steps that are required for each stage of construction. The manual emphasises
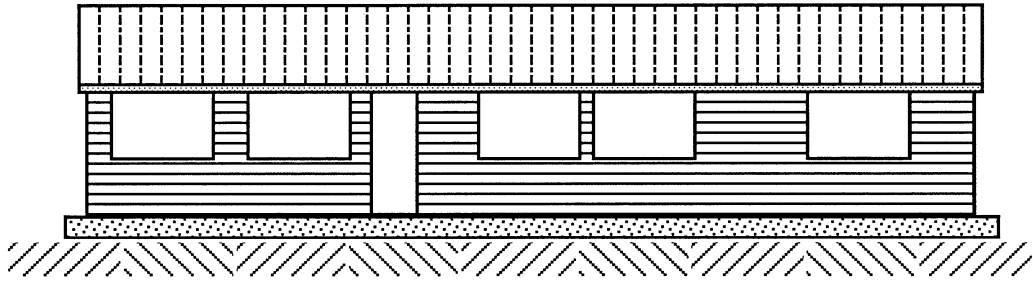
NORTH ELEVATION
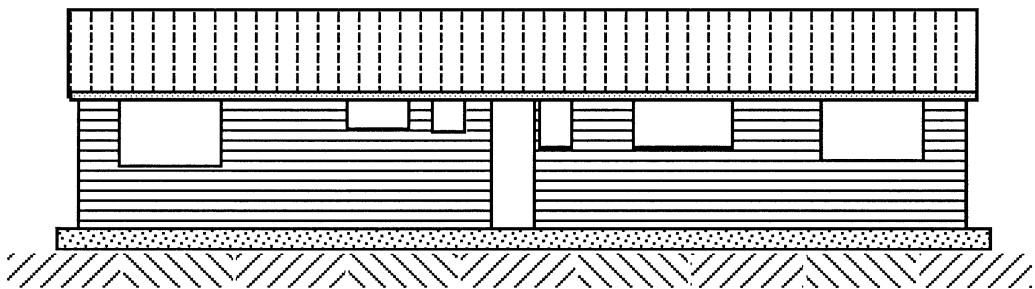
SOUTH ELEVATION
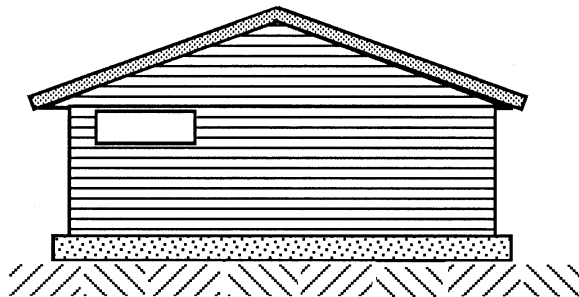
WEST ELEVATION

FIGURE 1   Elevations of the Test House

the importance of accuracy in laying out the slab, getting correct levels and casting-in the bolts for the external walls. The bottom plates of the external walls are drilled on site, to accommodate any slight misalignment of the bolts.

The house was erected by a specialist Nu-Steel fabricator on the Station's existing floor slab that had been used for previous test houses, but

FIGURE 2   House Plan with Approximate Room Dimensions

because of the greater width of the Nu-Steel house the slab had to be extended. Although the extension needed to be only 330 mm to produce the required 7500 mm width, it was decided to increase the width of the slab by 600 mm, 300 mm down each side. This allowed the cranked hold down bolts to be set in fresh concrete, as is normal practice, and also provided sufficient width for the new concrete to be suitably reinforced and tied into the existing slab.

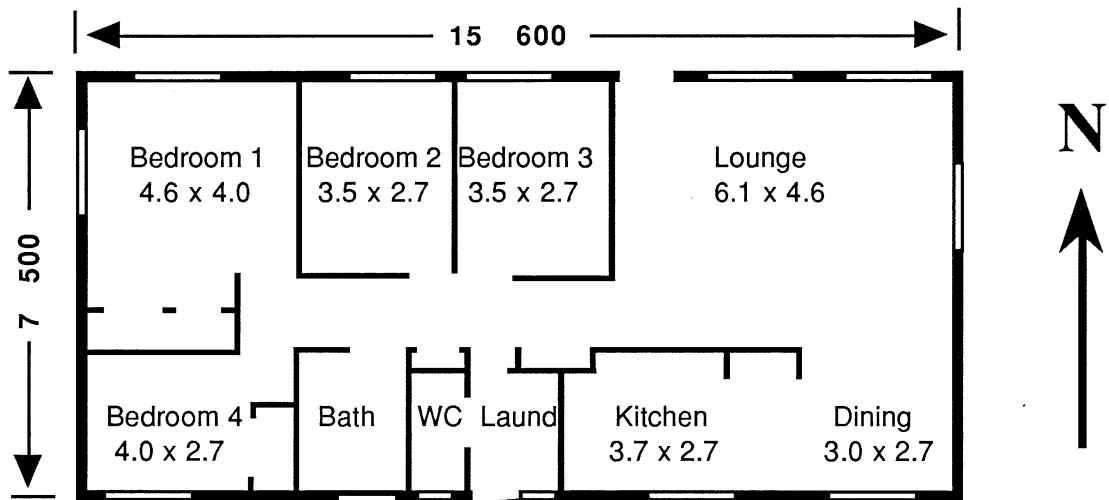As the existing slab was longer than required, the end walls were tied down with masonry anchors, rather than bolts set in new concrete. It was anticipated that this would cause no problems in the test programme.

The wall framing in the Nu-Steel system consists of Lysaght Building Industries' light gauge steel channel sections and stiffened channel sections while the roof trusses have Z-sections as principal members. A summary of components and construction details is given below.

Floor slab: The slab was 100 mm thick concrete cast over compacted fill. It had edge beams and internal beams that suited a previous house design, but did not line up with the walls of this test house. Its width was extended as described above.

Wall frames: The external wall frames were 2450 mm high and had studs at 600 mm spacing. The channel section components of the wall frames were made to fit snugly into each other. Thus the 75 x 32 x 1.2 mm studs fitted into the 78 x 31 x 1.2 mm bottom plate and the 72 x 34 x 1.2 mm noggings fitted into the studs at approximately mid height. The members were welded together at the joints. The top plate on the load bearing external walls was a 79 x 75 x 1.6 mm stiffened plate section

Truss stiffening plate

Hold down bolt

101 x 47 x 1.0 Z section

101 x 47 x 1.0 Z section

75 x 79 x 1.6mm Stiffened top plate

75 x 32 x 1.2 mm Stud

72 x 34 x 1.2 mm Nogging

78 x 31 x 1.2 mm Bottom plate

Floor slab
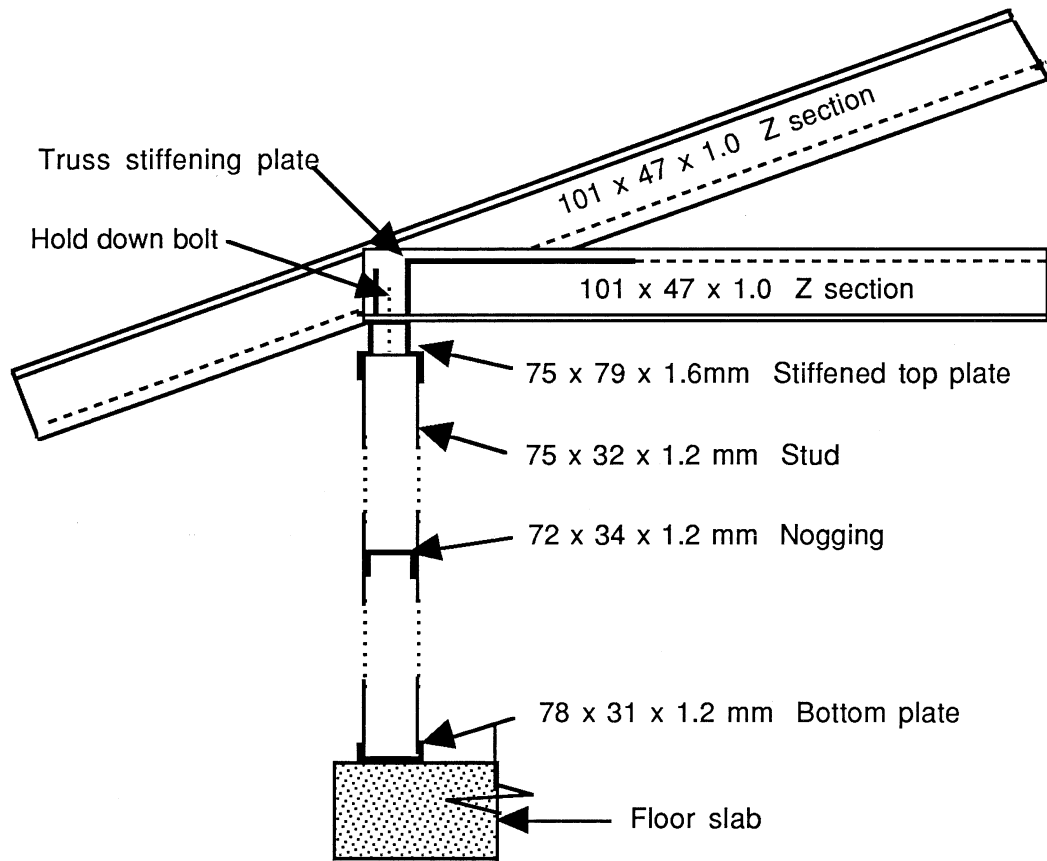
FIGURE 3   Typical Wall Section Including Truss

whereas that on the non loadbearing walls was the same section as the bottom plate. Figure 3 includes a typical cross section through an external loadbearing wall.

Double studs were used adjacent to window openings. They were welded together to form a 75 x 64 mm box section. Lintel beams over window openings along the load bearing walls were parallel chord trusses. They were 325 mm deep with the stiffened top plate as the top chord and a 78 x 31 x 1.2 mm bottom chord. The trusses had vertical stud members at 600 mm spacing and diagonal 75 x 32 x 1.2 mm members between studs. Figure 4 shows a typical lintel beam. The lintel beams for the non-loadbearing end walls were similar to the other lintels but did not have the diagonal members and the top chord was not a stiffened section.

The internal walls were of the same construction as the external non-loadbearing walls. The 78 x 31 x 1.2 mm channel section top plate meant that they were 50 mm shorter than the external load bearing walls, to allow for deflection of the roof trusses.

FIGURE 4 Typical Lintel Beam

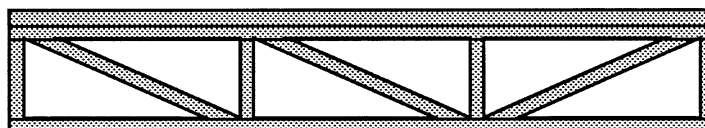There appeared to be plenty of weld at each of the joints in the frames. By the position of the weld, it was obvious that the frames had been welded from one side only. For example, at the stud/plate joints the rear leg of the stud had two vertical runs of weld and the front leg had one horizontal run.

External walls were braced with diagonal stud members welded between the nogging and the top or bottom plate member and extending across two stud spacings. The braces were aligned in opposite directions at each end of a length of wall. Hold down bolts were located adjacent to each brace/bottom plate junction. Internal wall bracing was provided by crossed diagonal 32 x 1.2 mm straps, pretensioned before being welded to the top and bottom plates. The straps extended across two stud spacings. Across the width of the house diagonal cross braces were fitted in the following walls: BR1/BR2, BR2/BR3, BR3/Lnge, Kit/Lndy, WC/Bath.

The bottom plates of the external walls were secured to the slab with cranked 10 mm bolts embedded in the concrete. Bolts were located at the ends of bracing members, adjacent to every opening and at a maximum of 900 mm spacing. Internal walls were fastened to the slab with 10 mm masonry anchors at 900 mm spacing, but internal bracing walls also had a two 12 mm masonry anchors set within about 50 mm of the end braced studs.

All internal walls were fabricated in one piece in the factory, but external walls had to be transported in sections. These sections were joined longitudinally with power driven 10 x 16 mm screws. At intersections of internal walls with external ones six such screws were used together with an M12 bolt at the top of the walls.

Roof trusses: The roof trusses were conventional W-braced trusses spanning 7500 mm with a 20° pitch. They had a 600 mm overhang and were spaced 1200 mm apart. The top and bottom chord members were 101 x 47 x 1.0 mm Z-sections while the braces were 75 x 32 x 1.2 mm channel members. All joints were welded. At the heel joint of the truss, the top leg of

the bottom chord had been cut back to fabricate the joint. This joint was then reinforced with a 100 x 75 x 6 mm U-shaped bracket welded in position. The bracket had a leg that extended approximately 300 mm along the bottom chord.

The gable end trusses were really triangular shaped framing rather than structural trusses. They did not have diagonal members but had vertical stud size members at 600 mm spacing to which the cladding was fixed. The top and bottom chord members were 75 x 32 x 1.2 mm, that is , the same section as the wall studs. The trusses sat directly over the end walls and were fastened to them.

The normal trusses were located directly over external wall studs or struts in the lintel beams and were fixed to the top plates with one M10 high strength bolt each end. A special device was attached to the nuts to prevent them from rotating during assembly.

Diagonal wind bracing was attached to the underside of the top chord of four trusses at each gable end. It extended from points adjacent to the ridge at the gable truss to the top plates about 4.2 m back from the gable end.

Battens: The roof battens were 0.75 mm thick LBI "top hat" section. Five battens were used on each slope resulting in end spacings of 900 mm and internal spacing of 1310 mm. The battens were fixed to the top chords of the trusses with four 14 x 22 mm self drilling screws per crossover.

The light gauge steel ceiling battens were of a similar shape to the roof battens but were made from thinner material. They were 0.47 mm thick. The ceiling battens were spaced 450 mm apart and were fixed with two 10 x 16 mm self drilling screws per crossover.

Cladding: The roofing was LBI Trimdek fixed to the battens with one 14 x 50 mm screw per rib. The screws were the coarse threaded Type 17 variety, which was developed for fixing to timber. The coarse thread may have been the reason for a number of screws being badly overdriven on the northern slope of the roof. This overdriving caused permanent dimpling of the roofing and in a number of instances even caused the roofing to split. This splitting would constitute a serious hazard in tropical cyclone areas. As the problem was restricted to the northern slope it was probably associated with a particular fixer.

The external wall cladding was 6 mm Hardiplank fixed to the studs with one 8 x 35 mm screw per crossover. Most of the internal wall lining was 10 mm plasterboard fixed in accordance with the manufacturer's specification. Water resistant board was used in the kitchen and laundry areas. The bathroom was lined with 6 mm fibre cement board. The ceiling lining was 10 mm plasterboard throughout.

3. LOAD SIMULATION AND RESPONSE MEASUREMENT

3.1 Determination of Wind Loads

The Nu-Steel framing system was designed according to the provisions of the Wind Loading Code, AS 1170 Part 2, to resist tropical cyclone wind forces when located in a suburban (terrain category 3) environment. The design details were not made available to the Station so that the test loads could be calculated totally independently. As the structural certificate issued with the house plans is dated 1984, it has been assumed that the design was based on the 1983 edition of the code. The test loads were therefore calculated from that edition. However as there has been a new edition issued in 1989, reference will also be made to it.

AS 1170.2-1983 (SAA,1983) lists the basic design wind velocity at a height of 10 m for tropical cyclone areas as 63.25 metres per second (m/s). This can then be modified for a height of 6 m and for terrain category 3 to become a design wind velocity of 42 m/s, which in turn can be converted to a free stream dynamic pressure of 1.05 kPa. This value was used to calculate the test loads.

Pressure coefficients for walls and for roofs of different slopes are given in the code. For pitched roofs different coefficients are listed for two orthogonal directions of approach of the wind. A decision had to be made therefore as to which direction of approach was considered to be the most critical. For wind blowing parallel to the ridge line the first four trusses would have very high uplift pressures acting on them. This pressure would be reduced on subsequent trusses until it became small. Figure 5 illustrates this effect in terms of pressure coefficients. Conversely, for the case of wind blowing perpendicular to the ridge line, the code recommends uniform but different pressures on each slope of the roof. For a 20° roof pitch the pressures would be in the order of 75% of the pressures on the end trusses. Figure 6 shows the pressure coefficients for wind blowing perpendicular to the ridge line.

From the information given above it may seem obvious that the former case should be the test case, however there are some constraints about that. Wind parallel to the ridge line would cause low racking stresses in the system, because the area of end wall is much less than that of the side
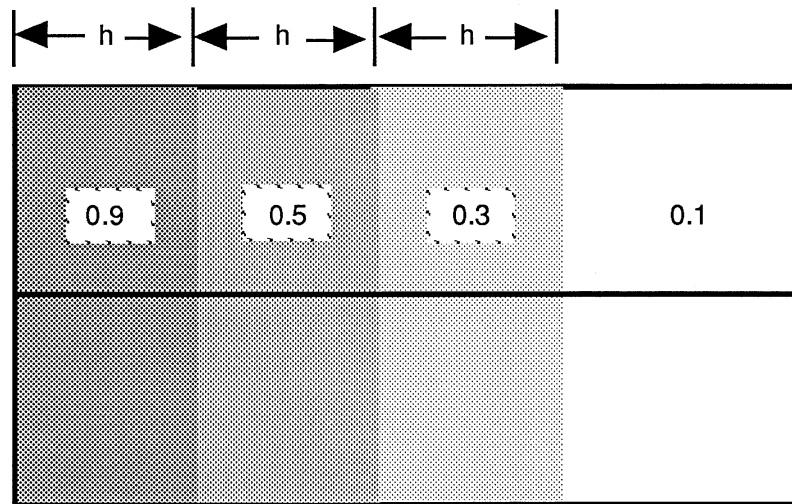
FIGURE 5   Pressure Coefficients for Wind Parallel to Ridge Line

walls and because of the length of wall available to resist the racking forces. Conversely, wind perpendicular to the ridge line would cause additional stresses on the truss hold down detail.

Because of the above argument, and because all of the trusses would have a chance of failing rather than just the end four, the test case was taken as the wind blowing perpendicular to the ridge line. However it was decided that if the system easily satisfied the loading from that direction, a loading sequence simulating wind parallel to the ridge line may be applied.

The Wind Loading Code allows the designer more freedom in the determination of internal pressure coefficients. They are based on the ratio of openings on the windward wall to openings on the other walls. One could accurately assume that windows and doors of a house are likely be closed during a tropical cyclone, especially the windward ones. However if there is flying debris about there is a strong possibility that a window will be broken and wind will penetrate the building, fully pressurising it. For this reason the Cyclone Testing Station recommends that the maximum internal pressure coefficient should be used when designing buildings for cyclone prone areas. An internal pressure coefficient of 0.8 was used in the calculation of the test loads. This is also shown on Figure 6.

3.2   Test Loads

Vertical reaction at the end of the trusses was taken as being the appropriate parameter to reproduce as exactly as possible with the simulated loading. This would mean that the axial forces in the trusses, the force on the truss hold-down detail and the uplift forces in the wall would be accurate. The pressure coefficients shown in Figure 6 result in
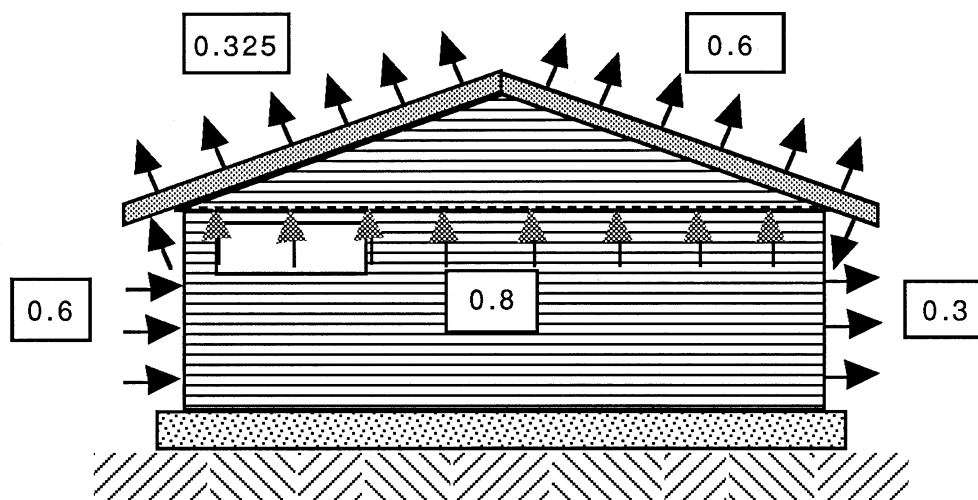
FIGURE 6   Pressure Coefficients for Wind Perpendicular to Ridge Line

almost equal vertical reactions at the end of the trusses. This is because of the effect of pressure acting on one overhang and suction on the other.

Figure 7 (a) shows the pressures that would be generated on the building for a design wind speed of 42 m/s, together with the reactions. It is very convenient from the test viewpoint that the two reactions are virtually the same, as it means that the loading shown in Figure 6 can be simulated by the same uniform uplift pressure acting on each slope. The uplift loading gear, described in Section 3.3, cannot apply different loading to the two roof slopes simultaneous with lateral loading. Also it applies the total uplift loading to the roof battens rather than to the roof and ceiling. Figure 7 compares the pressures acting on a truss from design calculations with those to be applied during testing.

Although Figure 7 (a) shows the internal pressure as acting on the ceiling, the loading rig has been designed to apply the total uplift load to the battens, as indicated in Figure 7 (b). This loading is considered quite valid as it represents the case when the building is pressurised internally; the manhole cover lifts and allows the internal pressure to act in the roof space. The internal pressure would then act on the underside of the roofing and transfer the forces to the battens.

3.3 Application of Loads

The loading system consisted of twelve large frames, six spaced evenly along each long side of the house. The forces were applied by means of hydraulic rams pulling on cables which were in turn connected to a load spreading system. All rams were connected to the same hydraulic pump, but controls allowed the pressure in the horizontal rams to be different from that in the vertical rams. This allowed the simultaneous application of independent uplift and racking forces on the house. Every effort was
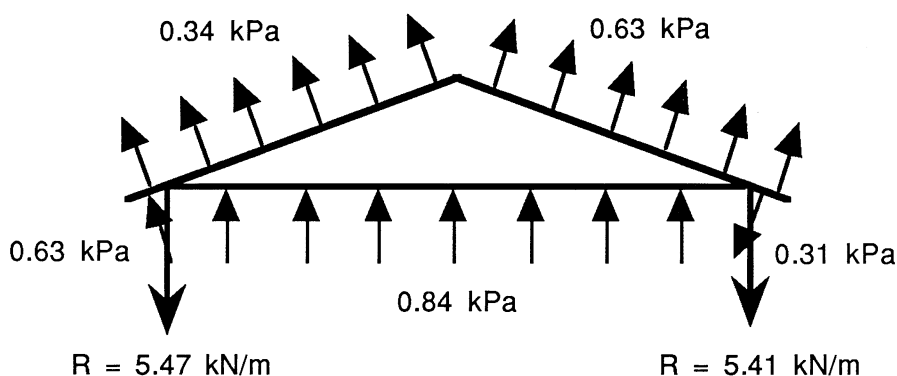
(a)   Pressures calculated for a design wind speed of 42 m/s
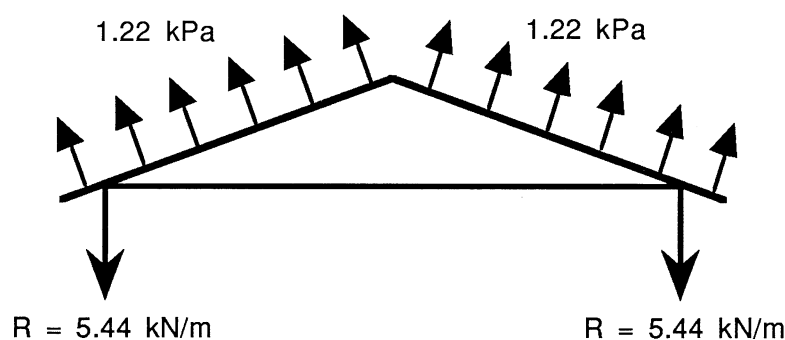
(b)   Test loading to produce nominally the same reactions

FIGURE 7   Loading on the Roof Trusses

made to ensure that the pressure in each ram of a particular group was the same.

During the test programme three different loading systems were used:

◊   Combined uplift and racking
◊   Uniform racking
◊   Individual wall racking

Each loading system involved at least one hydraulic ram pulling on a cable that was attached to part of the house.   The ram was usually loaded to a predetermined force measured by an electronic force transducer in series with the cable.   For static tests the load was increased incrementally.   At each increment horizontal and vertical displacements of the house were measured by electronic displacement transducers at numerous locations.

The different methods of load application will now be outlined.

3.3.1 Combined uplift and racking

Uplift forces simulating the combination of uplift pressure on the roof surface and internal pressure were applied to the roof structure by means of the twelve large loading frames, six loading each slope of the roof. Figure 8 illustrates the loading system. The hydraulic rams "a" pull down on one end of the large "see-saw" beams "b" causing uplift forces on load spreaders "c" attached to the roof. Each load spreader distributed the applied force over an area of 13.43 m^2, that is, the 4.63 m length of roof slope multiplied by the 2.9 m spacing of load frames. Each load spreading set reduced the applied load to sixteen equal portions which were then distributed to the underside of the roofing battens. Thus the loading system was capable of simulating uniform uplift pressure by applying a total of 192 uplift forces distributed evenly over the roof surface. Figure 9 illustrates the load distribution system. It should be noted that this method of loading does not impose any load directly onto the roofing. Although in this instance the decision was made because of loading constraints, it is accepted that the performance of roof sheeting under cyclic wind loading can be better assessed in the laboratory.
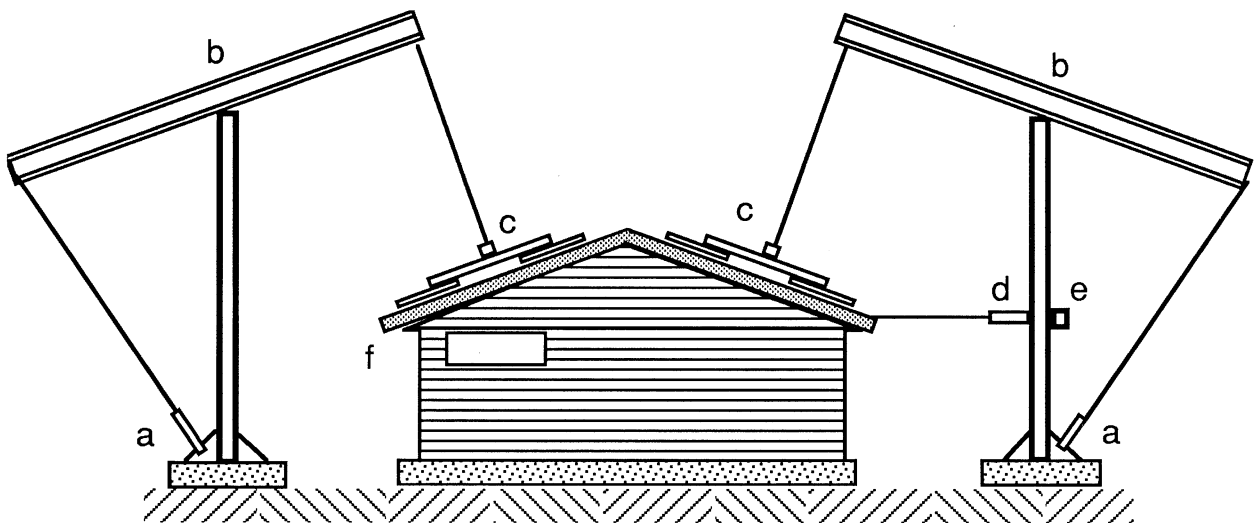
FIGURE 8   Loading System for Combined Uplift and Racking

Figure 8 also illustrates the system used to apply the horizontal racking forces. Four horizontally mounted rams "d" were attached to a large RHS steel beam "e" fixed to the uplift loading frames at wall height. A cable was extended above the ceiling from the ram to a load spreading system "f" at top plate level on the windward wall. Each ram load was distributed evenly to four loading points spaced 970 mm apart. Thus there were sixteen points of equal load simulating the uniform racking force along the top plate.

Combined uplift and racking forces were applied during the cyclic loading sequence. When incrementing the forces care was taken to maintain the correct ratio between uplift and horizontal pressures. The uniform racking force applied to the top plate was calculated as half of that caused by the sum of the pressure on the windward wall and the suction on the leeward wall, plus the total horizontal component of pressures on the roof slopes.

3.3.2 Uniform racking

When only the uniform racking forces were applied to the structural framework of the house, the method of application was exactly the same as for combined uplift and racking. That is, the uniform loading was simulated by sixteen concentrated horizontal loads spaced 970 mm apart along the top plate of the windward wall. This method of loading was used for some tests during construction of the house and for others during investigation of the bracing performance.

3.3.3 Racking individual walls

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

To rack a wall, a ram was aligned to the length of the wall and a cable was attached to the top plate of the windward wall at the junction with the wall to be racked.

3.4 Constraints on Loading

In most simulated loading programmes, compromises have to be made to accommodate the constraints imposed by the loading system or its ancillary equipment. Such compromises must be kept to a minimum, but if they are unavoidable an accurate assessment of their likely effects should be made.

Although it finally had only a minor effect on the applied loading, one of the major problems in setting up this series of tests related to the fact that the Nu-Steel house was of different dimensions from the previous test houses. In order to achieve the correct uniform distribution of uplift loading the large loading frames must be accurately located relative to the perimeter of the house. Thus to be totally accurate, either the frames had to be moved or some compromises made. Because of the high cost of relocating the frames the effects of compromise were examined and found to be minimal.

Because of the shorter length of the house, leaving the frames in the position used for previous test houses meant that the uplift load spreaders
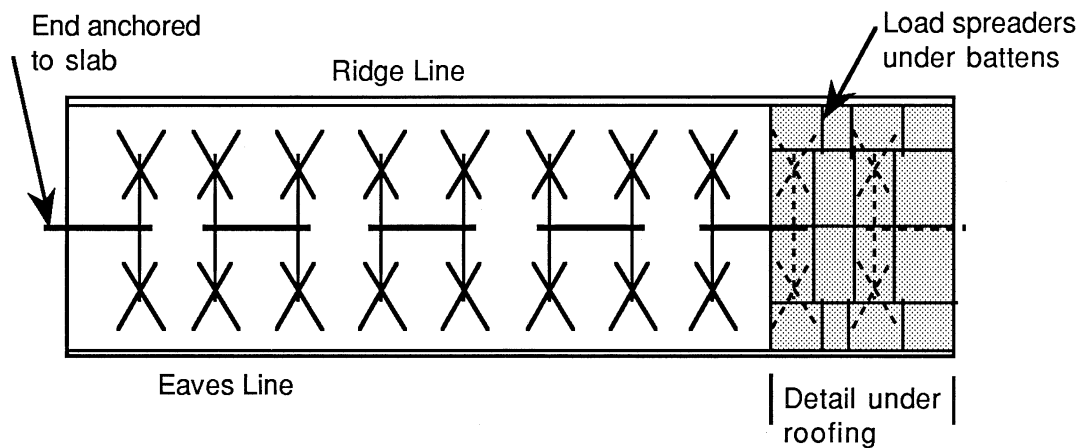
(a) Loading Arrangement
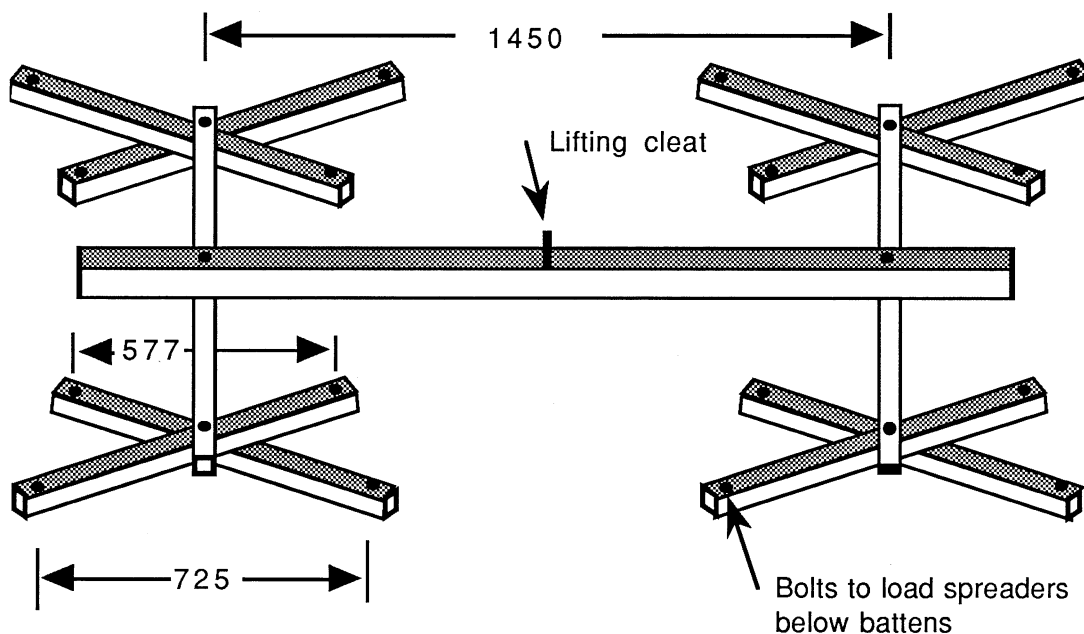
(b) Load Spreaders

FIGURE 9   Details of Uplift Loading System

at each end of the house would overhang the gable.   This meant that some device would have to be used to take the overhanging forces to ground.   It was not difficult to design such an arrangement.   Thus the house still had the nominally uniform loading along its length.   Figure 9 (a) shows the location of the uplift forces on one of the roof slopes.   However while this method of loading provided the equivalent of a uniform uplift pressure on the roof surface, its net effect was to apply along each slope 20 lines of loading from eaves to ridge.   This is also shown on Figure 9 (a).

The change in width of the house, together with the increased roof slope meant that the position of the uplift cable pulling from the "see-saw" beam to the load spreaders had to be changed to keep the force at 90° to the roof surface. Fortunately the correct position for the cable was along the beam length rather than beyond it. So, apart from having to fabricate some extra clamps, the change in width and roof pitch were accommodated without compromise.

The only other detail that had to be altered to suit this particular house was the geometry of the uplift load spreaders. Because of the change in batten spacing down the slope of the roof, the loadspreaders had to distribute different loads to each batten. This resulted in the asymmetric loading crosses shown in Figure 9.

As the trusses were spaced 1200 mm apart and the lines of loading were at 725 mm spacing, the loads applied to battens were sometimes close to the top chords. The worst case was at truss number 5 (numbered from the east) where the line load was within 100 mm of the top chord and thus this truss may have received a greater load than the average and the truss next to it may have received a smaller load. However the wide truss spacing relative to the spacing of the loading system resulted in reasonably even loading of the trusses.

3.5 Response Measurement

In order to interpret the behaviour of the house, accurate measurement of its response had to be made and recorded. As has already been stated, applied loads were monitored by electronic force transducers linked in series with some of the cables used to apply the loads. Displacements were measured by electronic displacement transducers and fed to a micro computer for processing and storage. The data acquisition system, which was specifically developed for the house testing research programme, has been described elsewhere (Boughton, 1983)

Displacement measurements were made at up to 60 locations on the house, depending upon the test being conducted. For the combined uplift and racking tests, vertical displacements at the heel of each truss as well as horizontal movement of the top plates were measured. Horizontal and vertical displacements were also measured at the bottom plate level for each transverse wall. All displacements were measured relative to ground via sets of independent scaffolding. The transducers were fixed to this datum by stands with magnetic bases, which allowed easy portability from one location to another if necessary. Figure 10 shows typical locations for measuring displacements of the roof and wall. Gauge "a" would be measuring horizontal displacement of the wall and gauge "b" would measure vertical movement of the truss.
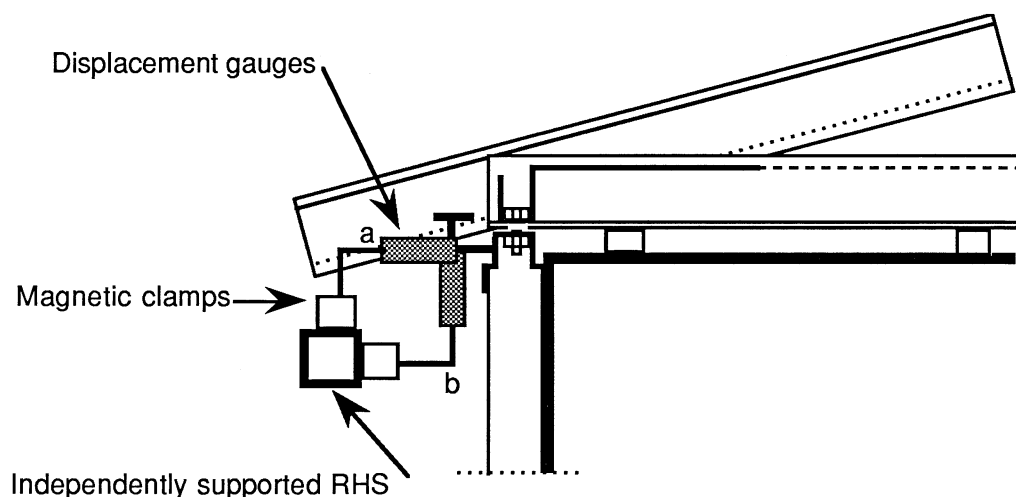
FIGURE 10   Typical Locations for Measuring Displacements

The load and deflection data were stored by the micro computer and transferred to magnetic tape on completion of the test. During the course of the test, the deflection at any point could be plotted against applied load and from that graph a determination made as to the likelihood of yielding of structural components.

4.   NON-DESTRUCTIVE TESTING

A programme of non-destructive testing was conducted on the test house during different stages of construction. The aim of these tests was to determine the change in response of elements of the house as the cladding and lining materials were installed. This change in response was measured as the difference in racking deflection of each of the transverse walls and of the lateral response of the house between these walls, at locations such as window heads. Therefore, as each type of cladding or lining was added a racking test was conducted on each of the transverse walls that joined the windward wall, and then a horizontal line load was applied to the top plate of the windward wall to simulate uniform wind loading of that wall.

The racking tests were conducted at the following stages of construction:

(a)   when the framing was complete and the walls had been lined

(b)   when the roof sheeting was added

(c)   when the ceiling lining was added

(d)   when the cornice was added

(e)   after the roofing screws had been removed.

The walls were not racked before any lining was installed as it was considered that the forces necessary to displace the top of the wall only 2 - 3 mm would have been too small to be applied accurately by the house testing equipment.   Incorporated in this was the concern of permanently deforming the frames by overloading them slightly.   The order of testing meant that the sequence in which the cladding was applied was different from normal practice.   The plasterboard was attached to the walls before the roofing was installed.

The final sequence of removing the roofing screws and thereby nullifying the effect of the roofing as a diaphragm was meant to give an indication of the performance of a house where the roofing did not have the capacity to act as a diaphragm.

The horizontal racking forces were applied to the top plate of the windward wall.   As previously shown in Figure 8 this was achieved by pulling on a cable or a series of cables installed above the ceiling.   The cables were attached to the stiffened top plate of the windward wall and were either in line with the particular transverse wall being tested or at 970 mm spacing to simulate the effect of uniform loading on the windward wall.   The cables were fitted over a pulley shape and bolted through the top plate.   To prevent a local bearing failure of the top plate at the bolt hole a piece of timber was fitted into it to transfer the bearing pressure to the side of the section.   Figure 11 shows the connection.
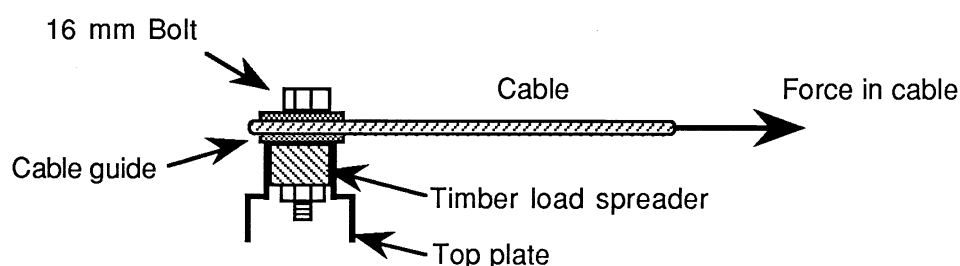
FIGURE 11   Racking Connection to Top Plate

For convenience the walls were numbered from the western end of the house, with the external wall being number 1.   The two external walls, 1 and 5, were braced with diagonal channel shaped stud members, two opposing braces per wall.   The three internal walls had flat strap cross bracing attached to one face.   The bracing had been tensioned.   The manufacturer's specifications list a racking capacity of 3.36 kN for the cross bracing but does not give a value for the channel bracing.   Apart from the three bedroom walls two other transverse walls were cross braced, the kitchen/laundry wall and the bathroom/WC wall.

Figure 12 shows the numbered walls, the braced walls (marked with a X) and the location of those gauges which measured any significant lateral movement. The gauge locations are circled. For the entire test series the wind was considered to be blowing from the north. Thus the north wall was the windward wall.

FIGURE 12   Position of Gauges for Preliminary Racking Tests

As the various cladding elements were added to the house higher racking loads had to be applied to obtain meaningful displacements. In the following tables these values have then been proportionately reduced for comparative purp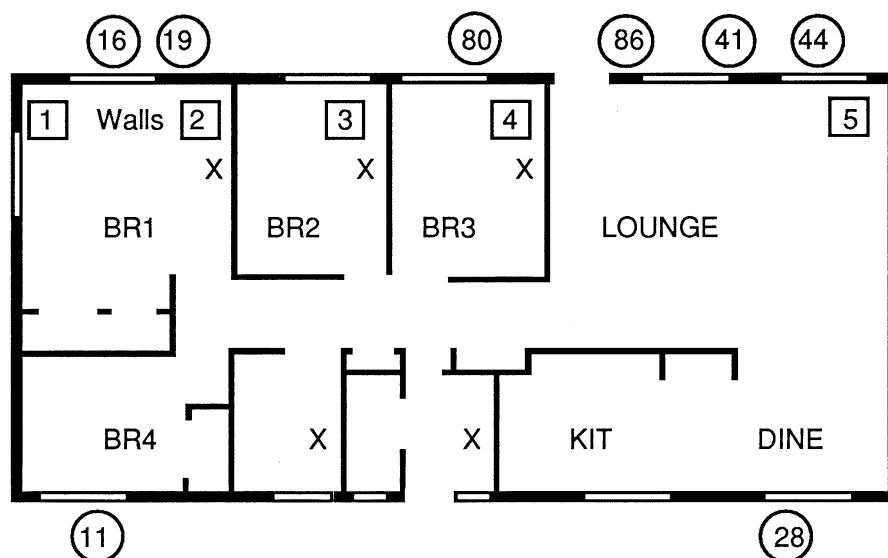oses. However both the accuracy of the displacement transducers and the absolute values of the displacement should be kept in mind when any comparisons are made. The displacement transducers are accurate to two tenths of a millimetre (0.2 mm). This means that there is virtually no difference between quoted values of say 1.1 and 1.2 mm. Also, the same degree of accuracy may not apply for very small absolute values of displacement, say less than half a millimetre.

For clarity Figure 12 lists only those gauges which are referred to in the following tables as measuring significant displacements. A total of 32 gauges were used to measure various responses during this series of tests. Some were located at the bottom of the transverse walls to measure any sliding or uplift due to overturning. None of these gauges measured any meaningful displacements in those directions. Even the walls that were not designated bracing walls, and therefore did not have extra restraint against the overturning component of the racking force, did not show any upwards displacement. This response was similar to that measured for timber framed houses that had been tested in previous programmes.

4.1 Racking of Individual Walls

At each of the stages of construction listed above, walls 1 to 5 were racked individually. The plasterboard was fixed to the internal walls in the normal manner only, that is it was not fixed according to bracing wall specifications. Despite this the plasterboard acted as a bracing medium and gave all of the walls a bracing strength well in excess of that specified by the manufacturer for the cross bracing system.

Table 1 lists the racking displacements for each wall during the series of tests. For ease of comparison the table lists the deflection of each wall at a racking load of 12 kN. During some of the later tests, however, the walls had to be loaded well in excess of that amount to obtain significant deflections. For example when the cornice had been installed, Wall 4 was loaded to 18 kN racking force to obtain suitable deflections. While there may be a few discrepancies between individual results there is no doubt that the roofing and ceiling act as membranes that can shed the load applied to one of the walls to other locations in the house and therefore reduce the racking deflection of that wall.

TABLE 1
RACKING DISPLACEMENTS OF WALLS LOADED TO 12 KN

House Configuration	Displacement (mm) at Walls				
	1	2	3	4	5
Walls lined	4.0	2.9	2.5	3.1	2.1
+ Roofing	3.4	2.6	1.4	1.6	1.5
+ Ceiling	2.9	3.0	1.5	0.9	1.6
+ Cornice	2.5	2.2	0.9	0.7	1.3
Roofing unscrewed	2.5	2.5	0.6	0.7	1.2

Wall 1 is possibly the best example of the reduction in deflection as the cladding was added. Its deflection decreased steadily as each element was added. The largest individual change occurred at wall 4 when the racking deflection was reduced by 1.5 mm after the roofing was installed. Wall 3 had a similar but slightly smaller reduction in racking displacement. It appears that the addition of the roofing membrane allowed these two walls to shed load to the cluster of walls in the wet area of the house.

As the cornice was glued to both the wall and the ceiling it provided a path for the transfer of forces through these elements. This path was particularly effective for the internal walls which had been specifically built 50 mm below the external ones. The cornice provided the final stiffening element of the system and resulted in a further decrease in racking deflection of each of the walls.

To simulate the effect of a roofing system incapable of acting as a diaphragm, the roofing was unscrewed. This configuration resulted in almost the same racking deflection of the walls as when the roofing was attached. That is, the ceiling and cornice were able to provide so stiff a path for force transfer that the presence or absence of the roofing made no apparent difference.

4.2   Uniform Racking

Although the uniform racking force caused some displacement of the transverse walls as well as the windward wall, the discussion in this section has been restricted to the response of the long walls of the house. Any conclusions relate to only to those walls and may not necessarily apply to the transverse walls.

The uniform racking tests, conducted at each phase of construction, graphically demonstrated the effect of the addition of each of the building elements. Before the roofing was installed the top of the windward wall was relatively flexible in the lateral direction, especially between transverse walls. Displacements in excess of 4 mm were measured at some locations. Table 2 shows the deflections measured at points along the windward and leeward walls at locations indicated in Figure 12.

It should be noted that the information listed in Table 2 is presented in a different form from that given in Table 1. In order to provide reasonable values for comparison the displacements have been given at different load levels. While the applied uniform racking load for the first test was approximately the design load, it had to be increased to three times that load to obtain meaningful deflections during the final tests. In order to compare the effect of the addition of each cladding element the displacements have been given at two load levels for all loads except the initial one. For example, when the roofing was added the house was tested to a uniform racking load of 2.0 kN/m but displacements in Table 2 are given for both 1.5 kN/m and 2.0 kN/m to enable a direct comparison with the displacements at 1.5 kN/m used for the previous configuration.

Table 2 shows how the roofing and the ceiling can act as diaphragms to transfer the applied force away from the windward top plate to the bracing walls and other parts of the house. This transfer of force is

TABLE 2
LATERAL DISPLACEMENTS CAUSED BY UNIFORM RACKING LOADS

House Configuration	Load (kN/m)	Displacement (mm) at Gauge Numbers							
		16	19	80	86	41	44	28	11
Walls lined	1.54	2.4	3.1	-	3.0	4.1	3.4	2.4	4.4
+ Roofing	1.54	1.5	2.2	1.3	1.6	1.4	1.2	1.1	0.3 (?)
	2.05	2.2	2.7	2.1	2.4	2.4	2.0	1.6	0.9 (?)
+ Ceiling	2.05	1.2	1.6	1.4	1.4	1.8	2.0	1.0	2.8
	2.56	2.0	2.9	2.2	2.2	2.5	2.6	1.4	3.6
+ Cornice	2.56	0.2	0.3	-	0.4	0.1	0.3	0.3	0.3
	4.62	1.0	1.6	1.7	2.6	1.6	1.4	1.2	1.0
Roofing unscrewed	2.56	0.4	0.7	0.6	0.7	0.4	0.6	0.2	0.3
	4.62	1.0	2.2	2.0	2.0	2.0	1.8	1.0	1.0

demonstrated by the significant change in deflection of the top plate as the various cladding elements are attached to the frame. For example the rate of displacement of the top plate measured by gauge 41 at the lounge room changes from 2.7 mm per kN/m of uniform loading to 0.4 mm per kN/m after all of the cladding is applied.

The table also illustrates the stiffening effect of the cornice as it binds the wall cladding and ceiling together. The addition of this element reduced the lateral displacements to about one tenth of their previous values, when there was ceiling only. A comparison of the two rows of displacements at a uniform loading of 2.56 kN/m illustrates this effect.

One other interesting aspect highlighted by this series of tests was the response of the leeward wall measured by gauge 11 at bedroom 4. With only the walls clad this gauge showed a surprising amount of movement of the top plate. A close inspection of the frame showed that the gauge was adjacent to a field joint in the external wall. Thus the top plate was not continuous at this location as field joints are made by screwing the webs of adjacent studs to each other. As the side of the joint measured by gauge 11 was some 900 mm away from the nearest truss there must have been

some magnification of the truss movement by the top plate. This situation continued until the cornice was installed, thereby preventing the independent displacement each side of the top plate joint.

In Table 2 the values for gauge 11 in the configuration with the roofing installed are considered suspect. At the time of testing it was thought that the installation of the roofing had prevented the top plate from undue movement at that location. But during the subsequent test the movement occurred again and was verified. It has been assumed therefore that the gauge malfunctioned during the previous test.

5. COMBINED LATERAL AND UPLIFT PRESSURES

This test simulates the effects of a tropical cyclone on the house. Uplift pressures are applied to the roof at the same time as lateral pressures are applied to the windward wall. To simulate the gustiness of a tropical cyclone the house was subjected to a series of load/unload cycles. The cyclic loading regime was that specified in EBS Technical Record 440 (EBS,1978). This requires a total of 10,200 uplift cycles to be applied to the roof structure combined with 1020 cycles of lateral loading. The cycles for each type of loading were applied in the following sequence:

(a)   uplift loading

       8000 cycles   0 - 5/8 design uplift pressure - 0

       2000 cycles   0 - 3/4 design uplift pressure - 0

        200 cycles   0 - design uplift pressure   - 0

    one application   2 x design uplift pressure

(b)   lateral loading

        800 cycles   0 - 5/8 design lateral pressure - 0

        200 cycles   0 - 3/4 design lateral pressure - 0

         20 cycles   0 - design lateral pressure   - 0

    one application   2 x design lateral pressure

The design pressures for the test were 1.22 kPa uplift and 0.95 kPa lateral. Allowance was made for the mass of the uplift loading gear when applying the uplift pressures. The constraint of having to apply the same pressure to each slope of the roof meant that the resultant lateral component of the actual design pressures had to be applied to the walls as an additional racking force.

The load cycling operation has been designed to be computer controlled. The computer was programmed to apply nine cycles of uplift load only, followed by a cycle of combined uplift and lateral load. This ensured that the two loading sequences were applied over the same time period.

Because of the relatively large volume of hydraulic oil that had to be

moved with each load cycle exerted by the twelve uplift rams the rate of cycling was much slower than would occur during a tropical cyclone. In fact the rate was approximately three cycles per minute. The computer control of the loading and measurement meant that the test could be left running unattended during the night. Because of the occasional failure of the hydraulic loading equipment, the cyclic load testing extended for about three days.

Although the cyclic load test is really a strength test without any limits on the amount of movement of the structure, displacement measurements were taken to give an indication of the performance of the house and to serve as a warning of distress of any element. In the final analysis of the test data, the displacement measurements taken during the load cycling are not given too much emphasis as their reliability is questionable. There can be no guarantee that the scaffolding, which acts as a datum for the displacements, has not been bumped by animals while the test is unattended, especially during the night.

Displacement measurements were made using a logarithmic sampling basis, taking readings frequently during the early stages of the test and less frequently towards the end. Sixty displacement gauges were used. They measured the vertical movement at each end of each truss, the horizontal displacement along the top of the windward and leeward walls and the sliding and overturning at the junction of the internal walls with the windward walls.

5.1 Wind Perpendicular to Ridge Line

5.1.1 Response to cyclic loading

The house appeared virtually unaffected by the 8000 cycles of uplift pressure combined with the 800 cycles of lateral pressure. Maximum displacements after the first cycle to 5/8 design pressures were about 0.5 mm vertically and 0.8 mm horizontally. By the end of the load cycling these displacements had increased marginally, to about 1 mm.

The first cycle of combined loading to 3/4 design pressures showed some unanticipated vertical displacement of the trusses near the north east corner of the house. The four trusses supported by the lounge room wall moved between 2 and 3 mm. By the end of the 2000 uplift cycles and 200 lateral cycles the vertical displacement had increased to about 6 mm at gauge 87 as indicated on Figure 13. It was also obvious that the displacement was not the result of relative movement between the truss and the top plate but was originating from the bottom plate. The cladding prevented any inspection of the bottom plate to determine the cause of the movement and it was decided not to disturb the cladding until the cyclic loading programme had been completed. The following day some of
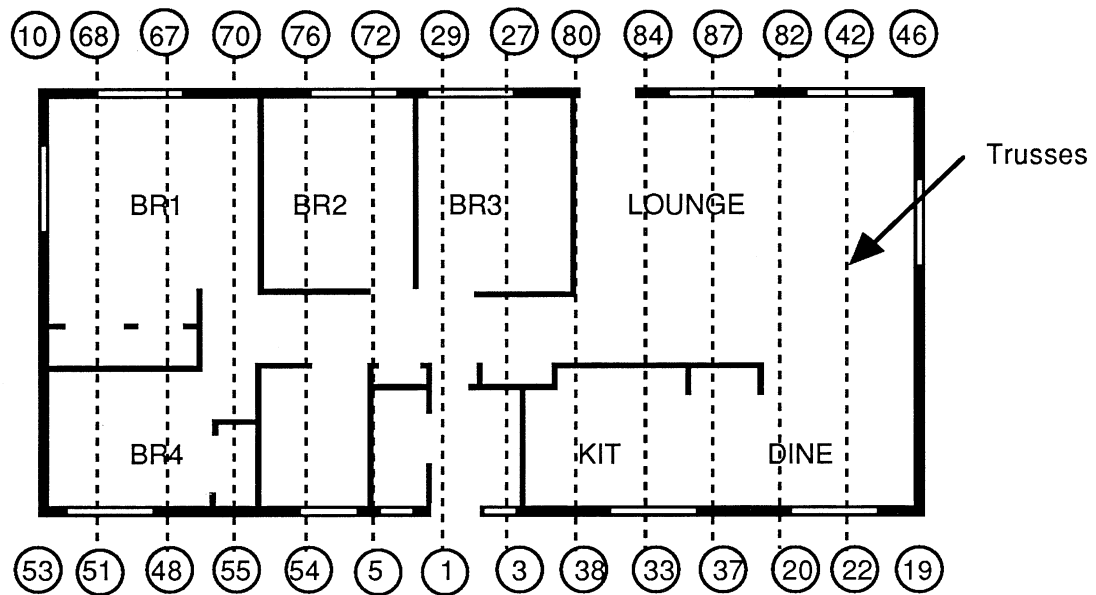
FIGURE 13   Position of Uplift Gauges for Combined Loading

the weatherboards were removed to show that the two bottom plate hold-down bolts between the windows on the north face of the lounge had started to withdraw from the concrete.

The first cycle of the combined design pressures caused more overall displacement of the house, with gauge 55 recording 2.5 mm vertical displacement and a number of gauges on the windward wall recording about 1 mm.   The cycling to combined design loads caused only about 4 mm displacement at gauge 87.   But as the gauges are reset between each series of cycles to a specific load level, the total withdrawal of the bolt could have been 8 - 10 mm.   The only other significant displacement was the upward movement of the ceiling.   In the centre of the lounge the ceiling, and thus the bottom chord of the truss, was moving about 6.6 mm upwards during each load cycle.

At the completion of the 10,200 cycles of uplift pressure and 1020 cycles of lateral pressure the only evidence of failure was the excessive deflection of the bottom plate below the lounge room window.   In itself, this displacement was not a cause for concern but an indication that the bottom plate was bending or tearing or that the hold-down bolt was withdrawing from the slab.   At this stage of the test programme it was still considered prudent not to breach or remove any cladding to determine the cause of the movement.

5.1.2 Static Overload

The final phase of the cyclic loading regime requires the application of combined pressures of two times the design uplift pressure and two times the design lateral pressure. For the Nu-Steel house this overload phase meant applying 2.44 kPa in uplift and 1.90 kPa laterally. The pressures were applied in twelve equal increments with displacements measured at each one.

To satisfy this overload segment the house must be capable of resisting the applied load without serious failure. The Nu-Steel house satisfied that criterion. The only sign of failure was the partial withdrawal of two of the hold-down bolts, as has already been mentioned. This did not prevent the house from resisting the prescribed overload of twice the uplift and lateral pressures. The maximum withdrawal distance of the bolts was about 10 mm.

The partial withdrawal of the bolts is probably related to the method of installation. As shown in Figure 14 (a) the hold-down bolts have a bent shaft to give them better holding power in the concrete. For the bent bolts to pull out they must break the concrete rather than just break the bond between the concrete and the surface of the bolt, as would be the case for a straight bolt. However the efficiency of the system is reliant on concrete flowing back into the void formed by pushing the bolt into the wet concrete. If the concrete starts to lose its flow characteristics before each of the 63 bolts is inserted in its correct location, a void such as shown in Figure 14 (b) can result. This would explain the partial withdrawal of the two adjacent hold-down bolts in the test house.
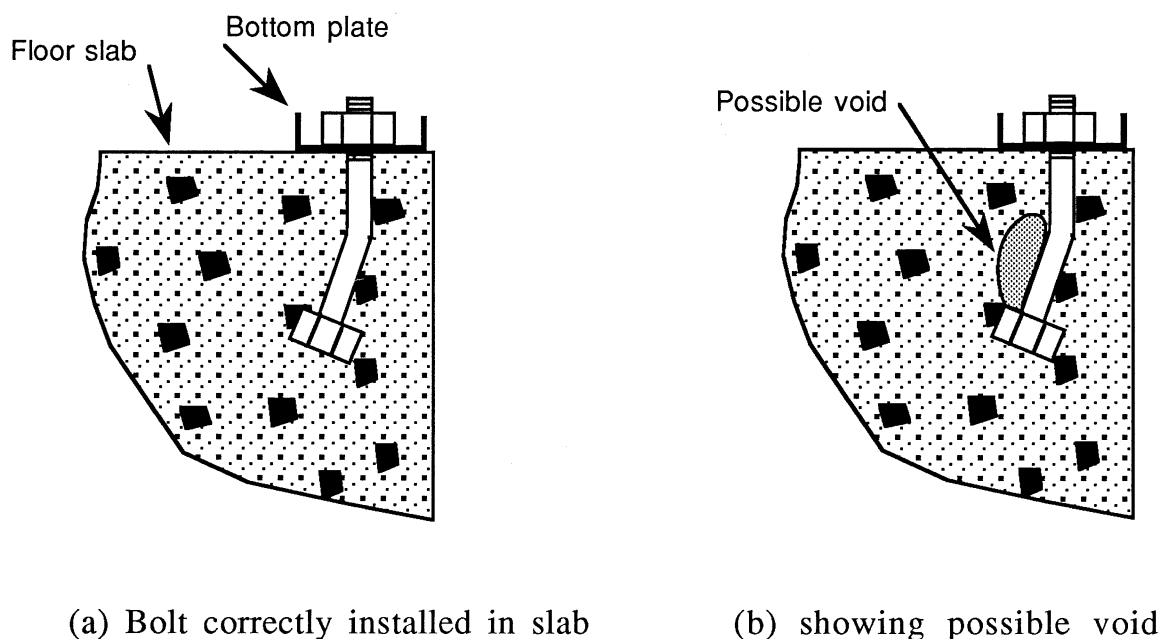

(a) Bolt correctly installed in slab   (b) showing possible void

FIGURE 15 Hold-down Bolt Details

TABLE 3
VERTICAL DISPLACEMENTS AT ENDS OF TRUSSES ON WINDWARD WALL

Load	Vertical Displacements (mm) at Gauge Numbers										
	10	68	67	70	72	27	80	84	87	82	42	46
1D	0	0.9	1.0	0.8	1.7	1.9	1.2	1.9	4.6	4.0	2.0	0.4
2D	1.5	5.3	6.4	6.0	7.4	8.4	9.2	9.4	13.4	13.9	13.7	2.3

During the static overload test the maximum upward displacement of the trusses over the two offending bolts was 13.4 mm at gauge 87, 13.9 mm at gauge 82 and 13.7 mm at gauge 42. While these three gauges reflected the withdrawal of the hold-down bolts, other trusses nearby also lifted a significant amount. Table 3 lists the total vertical displacement at the windward end of the trusses. The displacements are given at design load (1.22 kPa upwards and 0.95 kPa laterally) and twice design load (2.44 kPa and 1.90 kPa), listed as 1D and 2D in the table. The displacements are relative to ground and therefore are the sum of all the vertical components at each location. The positions of the gauges have already been indicated in Figure 13.

The values listed in Table 3 and in Table 4 for the leeward side of the house show that most of the displacement occurred between design load (1D) and two times design load (2D). The displacements at 2D are generally from five to seven times that for 1D. Figure 15 shows typical load displacement curves for some selected locations.
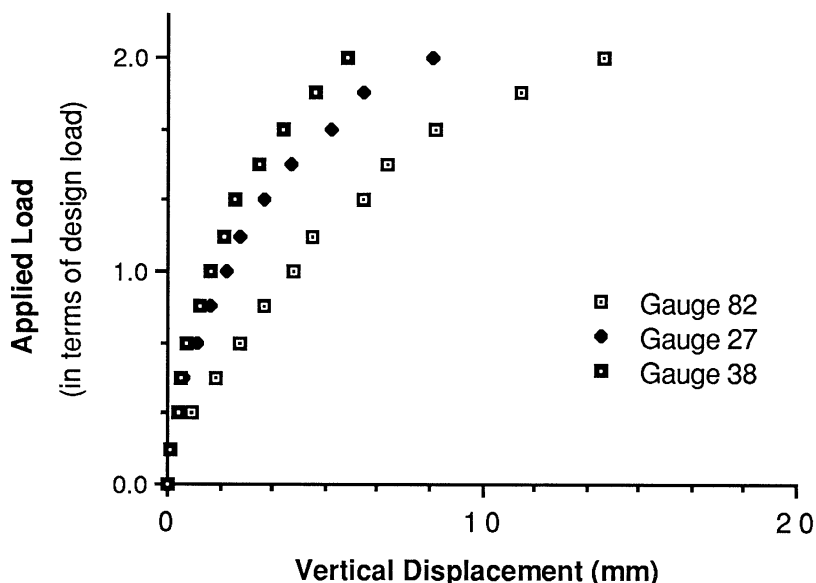

Figure 15   Typical Load/Deflection Curves

The withdrawing hold-down bolts appear to have contributed to the displacement of some of the adjacent trusses. Gauges 84 and 80 indicated the next highest displacements, 9.4 mm and 9.2 mm respectively. However it should be noted that all of the gauges along the windward wall measured greater displacements than those along the leeward wall. This reflects the vertical component of the overturning effect of the racking forces. In fact at a load of 2D each of the internal bedroom walls lifted approximately 3 mm at its junction with the windward wall.

TABLE 4
VERTICAL DISPLACEMENTS AT ENDS OF TRUSSES ON LEEWARD WALL

Load	Vertical Displacements (mm) at Gauge Numbers												
	22	20	37	33	38	3	1	5	54	55	48	50	49
1D	1.6	1.2	1.0	1.8	1.4	1.9	1.9	0.5	1.1	0.8	1.4	1.1	0
2D	4.8	6.3	5.7	6.8	5.7	6.4	4.5	3.2	4.5	4.5	5.3	4.3	0.9

There was no single obvious cause for most of the vertical displacements. The top plate did not appear to be bend significantly, although it was very difficult to observe. There was no excessive movement of the truss tie down. The only other member to show any obvious displacement was the bottom plate. On release of the overload, the bottom plate remained slightly bent between some of the hold-down bolts.

There was no evidence of the trusses over the bedroom areas shedding any of the uplift load through the cornice to the internal walls. They registered similar displacements to those free-span trusses over the lounge room area.

As indicated in the Tables, the gable end frames showed very little displacement at their end supports. This was to be expected as they were supported along their full length.

The major horizontal displacements are listed in Table 5. They were all on the windward face of the house. Because of some constraints on the test equipment, the horizontal gauges were not necessarily located in the same

positions that they were in the preliminary tests. Figure 16 shows the location of the principal horizontal gauges for the cyclic and overload tests.

The maximum horizontal displacement of 6.1 mm at twice the design load was measured by gauge 41 above the location where the bolts were withdrawing from the slab. This is probably fortuitous as that location

TABLE 5
LATERAL DISPLACEMENTS

Load	Lateral Displacements (mm) at Gauge Numbers									
	13	11	64	65	77	28	83	41	44	47
1D	-	1.3	0.0	0.3	0.0	0.8	1.3	0.6	0.7	1.0
2D	2.9	4.6	1.8	2.7	2.8	5.8	4.8	6.1	5.0	2.9

was also about midway along the largest distance between bracing walls. Gauge 28 at the window in bedroom 3 measured 5.8 mm at that load. As with the vertical displacements, the horizontal displacements increased significantly after the application of design load.

The most noticeable aspect of the lateral displacements is that they are all much larger than were recorded during the preliminary loading tests which have been listed in Table 2. The applied overload pressure of 1.90 kPa for this test can be converted to 3.1 kN/m for comparison with the values in that table. It is not clear whether the additional displacement is due to the effects of cyclic loading or the combined uplift.
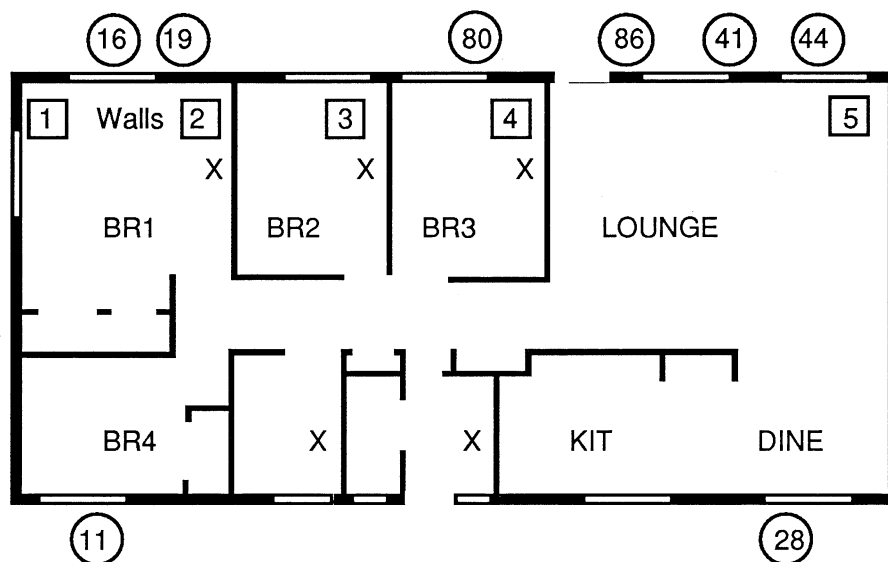

FIGURE 16   Location of Major Lateral Gauges for Overload Tests

5.2   Wind Parallel to Ridge

Because the test house was still virtually intact at the completion of the test programme for wind perpendicular to the ridge line, it was decided to investigate its likely performance for winds acting parallel to the ridge. Wind blowing parallel to the ridge can cause high uplift pressures at the end of the house, but they gradually reduce in intensity along the length of the roof.   The Wind Loading Code has simplified the distribution for design purposes.   This simplified distribution of pressures has been illustrated in Figure 5 in terms of pressure coefficient.   For the test house, the end 4.4 m of roof would have a pressure coefficient of 0.9 and the next 4.4 m would have a coefficient of 0.5.   Figure 17 shows the overall pressures acting on the trusses in these zones.

0.95 kPa   0.95 kPa

0.63 kPa   0.63 kPa

0.84 kPa

(a)   End 4.4 m length

0.53 kPa   0.53 kPa

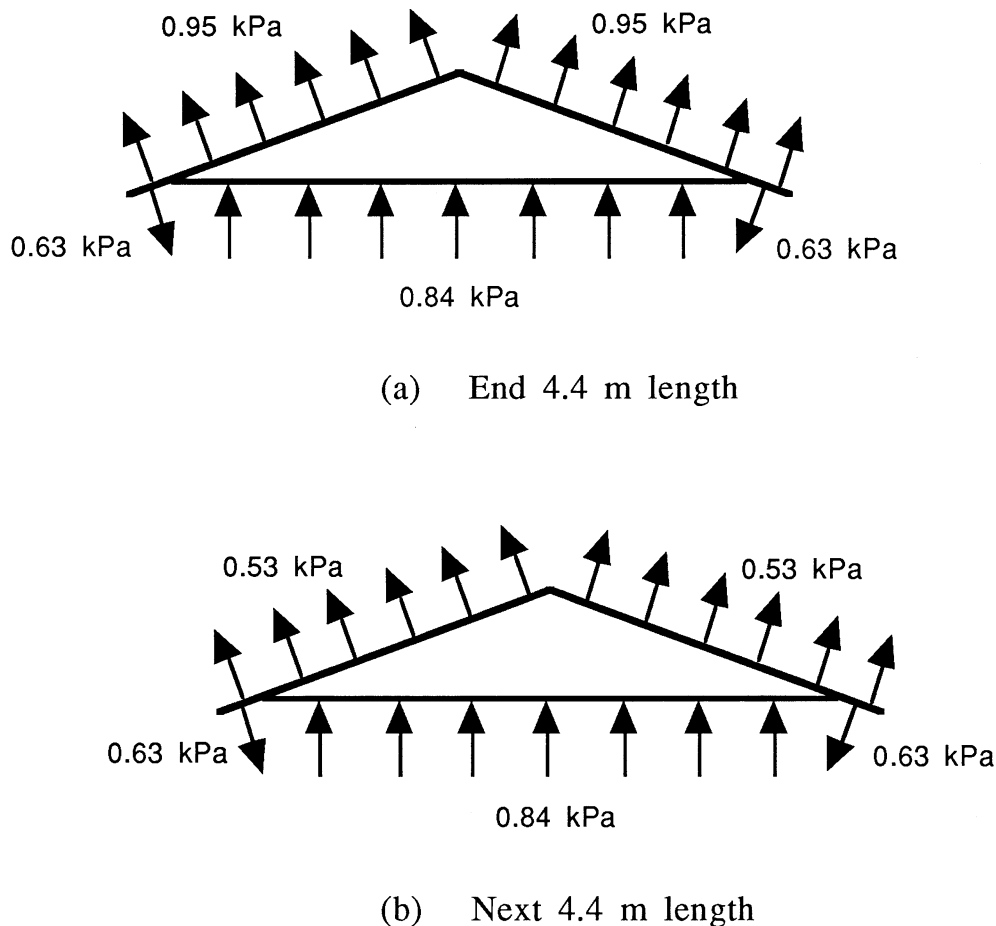0.63 kPa   0.63 kPa

0.84 kPa

(b)   Next 4.4 m length

FIGURE 17   Pressure on Roof for Wind Blowing Parallel to Ridge

In order to simulate the uplift pressures for this type of loading a number of compromises had to be made.   Firstly, no racking forces were applied to the house.   This was because the end elevation did not present a large area for the wind to blow on, and the longitudinal external and internal walls indicated significant racking strength.   This decision meant that the two

hydraulic circuits could be used to apply different uplift pressures to the roof, allowing simulation of the end zone pressure coefficient of 0.9 and the adjacent zone with its pressure coefficient of 0.5.

The second compromise was that the test areas were not exactly 4.4 m long, that is, the height of the building. The actual distances were governed by the spacing of the large loading frames. Two frames on each side of the house loaded the end area and another two loaded the next area. Thus the higher loaded end area was 4.8 m long and the next area was 5.8 m long. This compromise was considered of little consequence.

The other compromise was in making an allowance for the effect of the previous cyclic load test. It was decided not to apply the 8000 cycles of 5/8 design uplift pressure, but to start the test with 2000 cycles of 3/4 design uplift pressure. There was no valid engineering basis for this decision except that it offered a simple and convenient compromise. It would have been very conservative to apply the whole loading regime again to the house which had already been loaded to 10,200 cycles of loading followed by an overload of two times design load. Also the saving of the time necessary to apply the 8000 cycles was very attractive.

It must be stressed that convenience was the main reason for this decision. There is no intention to imply that that the previous loading pattern can be considered equivalent to 8000 cycles of 1.0 kPa of uplift. Application of the 8000 cycles of load is rather tedious and can be frustrating because of the inevitable equipment failures that occur.

If the house had remained undamaged during the previous test, the second cyclic loading sequence would have been applied to the lounge/dining room end of the roof, because of the clear span of the trusses. But because of the partial withdrawal of the hold-down bolts in this area the loading had to be applied to the bedroom end of the house. This did not cause any concern, as the previous tests showed that the walls did not provide these trusses with any structural advantage over those at the other end of the house.

5.2.1 Response to Cyclic Loading

The application of the 2000 cycles of uplift pressures caused some bending of the bottom plate of the outside wall of bedroom 4. It was lifting about 3.5 mm which resulted in the ceiling lifting about 6.7 mm. There were other smaller deflections but in general the house had little trouble in resisting the loading.

The 200 cycles to design pressure caused a similar but greater response. The ceiling in bedroom 1 and in bedroom 4 lifted 8.5 mm and 8.0 mm respectively. Even in the lounge, which was at the unloaded end of the house, the ceiling was lifting 3.5 mm. But the house satisfactorily resisted

the full complement of load cycles.

5.2.2 Static overload

The roof structure was loaded incrementally towards the target pressure of twice design load. However at 1.87 times design load failure occurred as some battens pulled off the rafters. On the south slope of the highly loaded end area of the house, three of the five battens pulled off seven trusses. Presumable one batten rafter connection failed first, causing the adjacent ones to become overloaded in a domino effect. Neither the ridge batten nor the eaves batten pulled away from the trusses.

The failure mode was by the four 20 mm long 14 gauge screws bursting out of the 1.2 mm steel top chord. A detailed inspection of the failures showed that in many instances cracks emanated from the screw holes. It is not known whether these cracks were initiated during the cyclic loading regime or if they were a result of the screws bursting through the top chord.

Because of the type of failure an analysis was made of the force distribution from the batten into the top chord, to check if the method of load application had caused an unintentional overloading of the battens.

5.2.3 Assessment of joint strength

Figure 18 shows the assumed uniform loading case for wind on one slope of the roofing. Obviously, the batten spacing causes the the highest uplift loading to be on the central batten. A theoretical analysis of this uniform loading case, for trusses spaced 1200 mm apart, shows the reaction at the central batten to be 2.66 kN at the third truss from the west wall.
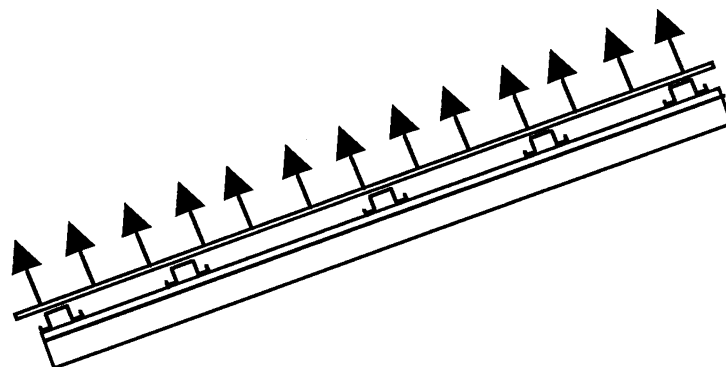

FIGURE 18   Assumed Wind Loading on One Roof Slope

The actual loading imposed on roof slope is shown in Figure 19 (a) and the resultant loading on the central batten is shown in Figure 19 (b). A

detailed analysis of the distribution of these point loads shows an uplift reaction of 2.67 kN, which is in remarkable agreement with the theoretical reaction. Thus the simulated loading system did not cause the batten rafter connection to be overloaded.
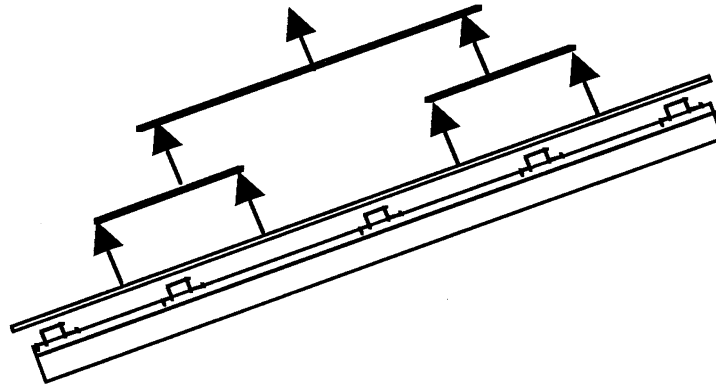
(a)   Loads Applied to Roof Slope

Truss numbers   Applied loads
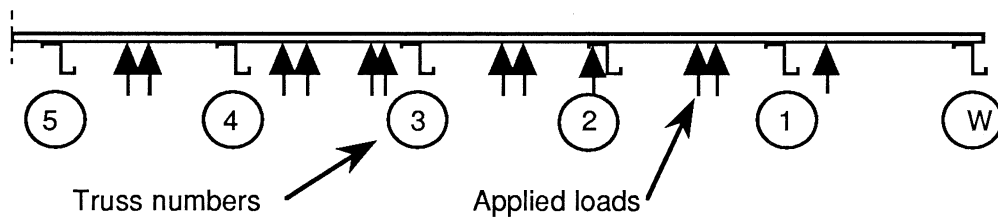
(b)   Loads Applied to Central Batten

FIGURE 19 Test Loads Applied to Roof Members

Based on the above analysis, the load that caused failure at the batten/ rafter interface was 5.0 kN. Assuming an even distribution of load between the four fasteners at the joint, a load at failure of 1.25 kN per fastener can be calculated. This load is well below the minimum axial withdrawal force of 3.1 kN specified for 14 gauge screws in 1.6 mm steel (Standards Australia, 1988). Although no value is given in the code for 1.2 mm thick steel, one could expect that it would not be too much below the value for 1.6 mm thickness. In some early literature, one manufacturer published a graph showing a pullout strength of approximately 3 kN for 1.2 mm thick steel (Deutsher, 1980). Even allowing for an uneven distribution of load among the four screws in the joint, a failure load of 1.25 kN is hard to justify. It must therefore be concluded that the joint strength was affected by the load cycling.

It should be noted that the test programme and the above analysis have included the internal pressure to be acting on the underside of the roofing. In practice this would happen when building was pressurised because of a dominant opening on the windward wall, causing the manhole cover to lift and the roof space to become pressurised. However if the house does not have access to the roof space, or if the cover can be locked to prevent it from dislodging, the pressure may not act on the underside of the roofing. Or it may be delayed so that its peak does not coincide with the peak suction on the external surface.

5.2.4   Comparison with AS 1170.2-1989

The 1989 edition of the Wind Loading Code (SAA, 1989) represents the best currently available knowledge on wind speeds and their effects on buildings. For most of the Australian coastline subject to tropical cyclones, with the exception of a small area in Western Australia, the basic wind speed for permissible stress design is 57 m/s. As the code no longer requires a cyclonic multiplier this wind speed would compare with 63.25 m/s (55 x 1.15) calculated from the 1983 edition of the code. Allowing for small changes in some other parameters the basic wind pressure on the test house would be reduced from 1.05 kPa to 0.95 kPa.

The net effect of this is to reduce the pressure on the rafters from the 0.95 kPa shown in Figure 17 (a) to 0.86 kPa. The internal pressure would reduce from 0.84 kPa to 0.76 kPa and the eaves suction would reduce from 0.63 kPa to 0.57 kPa. These reductions would mean that the load at which the batten screws pulled out would be the equivalent of 2.07 times the design pressure on the end trusses. Thus in terms of the 1989 edition of the code, the test house would have satisfied the overall criteria for wind acting parallel to the ridge.

Although the overall structure of the building may have been satisfactory the joint that failed should really have been designed for high local pressures that can occur on small areas of roofing. They attract a multiplier of 1.5 or 2.0 on the negative pressure acting on cladding and its immediate supporting members and fasteners. For the test house, with a planform width of 8.7 m, an area of cladding 1.74 m square should be designed for 1.5 times the negative external pressures plus any internal pressure. This area is larger than the tributary area of the joint that failed so that in practice the joint could be subjected to even more severe loading conditions than were used in this test programme.

6.   EXPLORATORY WALL RACKING

The object of this series of tests was to investigate the effect of systematically disabling the specified bracing system for the internal walls and, later, nullifying them completely by removing the lining material. A

uniform racking test was conducted before the programme started and between each step in the dismantling procedure.

The latter tests were to investigate the effect of internal wall spacing on the lateral strength of the house. Queensland building regulations specify a maximum spacing of bracing walls of 9 m for terrain category 3 of cyclone areas.

The test pattern was to simulate the racking forces by applying a uniform lateral load to the top plate of the designated windward wall. The lateral response of the house was measured by 32 displacement transducers located mainly along the top plate of each long wall. The gauges were positioned to measure the response of the transverse walls and the windward and leeward walls between the transverse walls. Figure 20 shows the location of the more important gauges as well as indicating the bracing walls. From the previous tests it was anticipated that there would be little measurable response of the house at design pressure. It was therefore decided to apply the load in increments up to 2.7 times the lateral design pressure in the first instance and reduce the load for later tests if necessary. No uplift pressure was applied as the failure of the batten joints precluded this. That failure also meant that the roofing would be unlikely to function as a diaphragm, however earlier tests had indicated that the ceiling had the capacity to do that on its own.
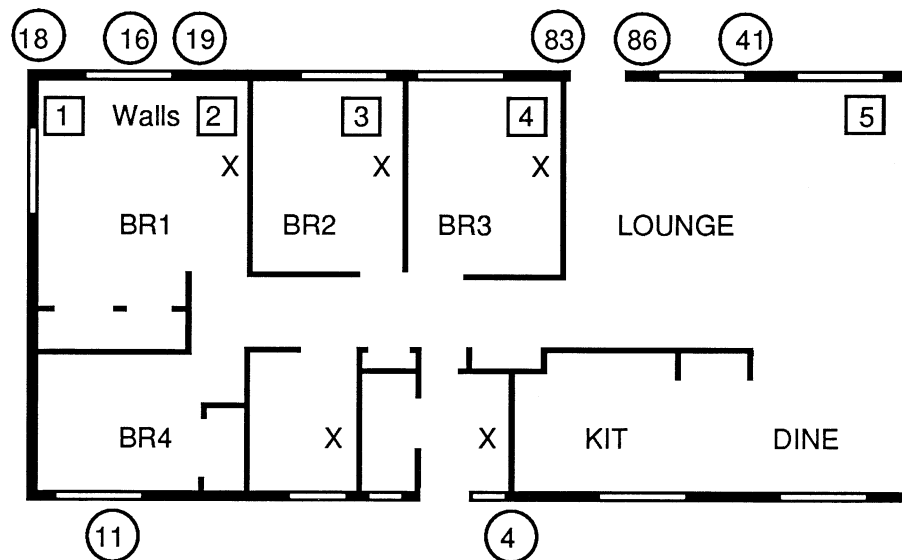

FIGURE 20   Location of Bracing Walls and Important Displacement
Transducers for Exploratory Wall Racking Tests

6.1   Disabling the Diagonal Bracing

Five of the internal wall frames were braced with crossed diagonal steel strap members, their location has been indicated on Figure 20. The pattern for this series of tests was to cut the two braces in one wall and then apply
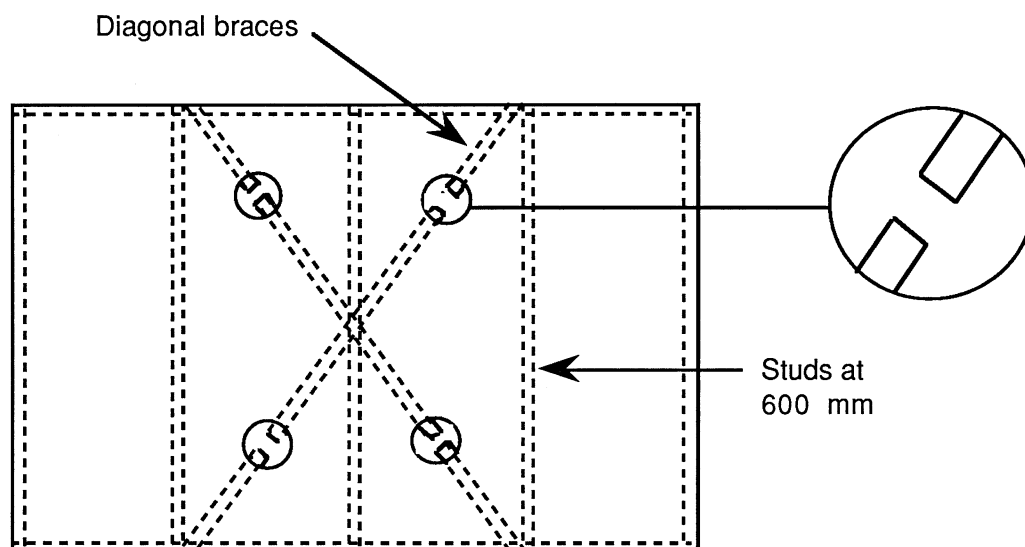
FIGURE 21   Disabling Wall Braces

a uniform racking load to the top of the windward wall to observe any change in response of the house. As a basis for comparison, a racking test was also conducted before any of the braces were cut. When cutting the braces between each stud, care was taken to ensure minimal damage to the plasterboard. Figure 21 shows the technique. The braces in Wall 4 were cut first followed by wall 3 and then Wall 2 after which all of the other braces were cut.

Most of the lateral movement occurred along lintels over the window openings. The maximum movements were comparatively small, only in the order of 2 to 4 mm. Table 6 summarises the major displacements measured during the test series.

The results given in Table 6 should be put into perspective in two respects. Firstly, in order to achieve meaningful lateral displacements the house was loaded to 2.7 times its design lateral load for cyclone wind conditions. Therefore the displacements at design wind loads would be less than one third of the values listed in the table, as the load displacement curves were not linear. Secondly, the displacement measuring equipment had been developed for significantly larger displacements and therefore had an accuracy limit of 0.2 mm. Thus the difference between, say, 1.2 mm and 1.4 mm may not be significant. The main information contained in Table 6 therefore can be summarised as follows:

(a)   the house was very stiff and strong under lateral loading

(b)   the internal lining material provides a much more effective bracing medium than conventional diagonal bracing.

TABLE 6
DISPLACEMENTS MEASURED AFTER CUTTING DIAGONAL BRACES

Braces cut in Wall Numbers	Displacements (mm) at gauge numbers							
	18	16	19	83	86	41	4	11
None	1.6	1.4	4.4	1.9	2.6	2.6	1.0	1.2
4	0.5	1.2	2.5	1.1	1.6	0.5	0.5	0.4
4, 3	0.6	1.0	1.2	0.8	2.1	0.8	1.2	-
4, 3, 2	0.4	0.8	1.4	0.7	1.4	0.4	0.8	0.8
All	0.4	0.8	1.6	0.8	1.3	0.4	1.2	0.9

As can be seen from the gauge positions shown in Figure 20, almost all of the major lateral displacements occurred on the windward face. The only significant movement on the leeward face was measured by gauges 4 and 11. This lack of movement on the leeward face was not unexpected as it showed that even without their diagonal braces the transverse walls were still acting as bracing walls and transferring most of the lateral forces to ground.

A further point illustrated in Table 6 is that there was virtually no extra displacement at the west end of the house where the roof batten joints had been damaged from the cyclic loading segment of the test programme.

6.2   Removing Plasterboard Lining

The next group of tests in the exploratory racking series involved the systematic removal of the internal lining from the transverse walls. The lining was first removed from both faces of wall 4 and the house was tested. This process was repeated until the lining was removed from internal walls 2, 3 and 4. In order to simulate continuity of the ceiling, a strip of plaster 100 mm deep was left at the top of each face. It was still glued to both the cornice and the wall studs and would therefore reasonably simulate continuity of the ceiling as if no wall had been there.

Once again the house was loaded to 2.7 times the design lateral loading for cyclonic wind conditions during each test, to ensure that significant displacements occurred. The major displacements measured during the systematic removal of the wall lining are listed in Table 7. The reference
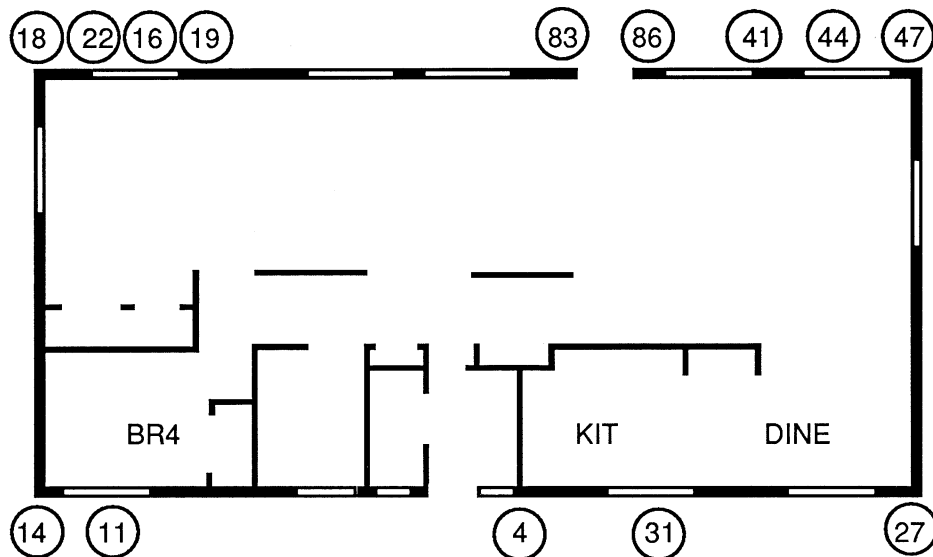
FIGURE 22   Effective Floor Plan and Gauge Locations

test, the first listed in the table, is for all diagonal braces cut but before any plasterboard had been removed.   Thus it is a repeat of the results of the last test listed in Table 6.   Three of the gauges that are listed in Table 7 recorded insignificant movement during that reference test but showed some movement as the plasterboard lining was removed from the walls. An effective plan of the house after removal of the plasterboard from the walls together with the location of the gauges is shown in Figure 22.

As can be seen from Table 7, the nominal removal of the three bedroom walls had only a small effect on the lateral response of the house.   The

TABLE 7
MAJOR DISPLACEMENTS DURING REMOVAL OF WALL LINING

Lining Removed from Wall Numbers	Displacements (mm) at Gauge Numbers													
	18	22	16	19	83	86	41	44	47	27	31	4	11	14
None	0.4	1.0	0.8	1.6	0.8	1.3	0.4	-	-	1.0	-	1.2	0.9	0.8
4	0.4	0.7	0.8	1.0	1.2	2.2	0.8	0.4	0.5	1.2	0.4	1.1	0.7	0.7
4, 3	0.9	1.4	1.3	2.0	1.6	2.4	1.1	0.4	1.0	2.2	0.7	2.4	0.5	1.4
4, 3, 2	0.8	1.1	1.2	1.6	1.7	2.1	1.1	0.8	0.7	2.1	0.3	1.9	1.4	1.2

maximum displacement of the windward wall was only about 2 mm. Even at the west end of the house where the roof batten joints had been broken, the maximum displacement was only 1.6 mm. This performance indicates that the ceiling must have been acting as a rigid diaphragm in transferring the lateral forces to the remaining internal walls in the bathroom/laundry area of the house. These walls still acted as bracing walls despite the fact that they had their diagonal braces cut during an earlier test. This theory is reinforced by the amount of movement of gauges 4 and 11 on the leeward wall.

7   LATERAL LOADING OF LINTEL

At the request of the manufacturer a test was conducted on one of the 1800 mm lintels in the house. The lintel over the dining room window was chosen because it was undamaged and it would not have been affected by the batten failure or by the partial withdrawal of the hold-down bolts.

The primary aim of the test was to measure the lateral displacement of the lintel beam under tropical cyclone wind conditions. It was assumed for test purposes that the lintel was actually supporting a sliding window extending from floor to the lintel beam and thus it would have to resist half of the total pressure acting on the window.

The window was assumed to be 2100 x 1870 mm and located near to the corner of the house. It would therefore attract a local pressure factor of 1.5 on the external suction forces. An internal pressure factor of 0.8 was also assumed to act concurrently with the external pressure. This resulted in a design pressure of 1.79 kPa acting on the sliding door. To simulate this pressure a pair of point loads was applied to the bottom chord of the lintel, nominally at third points. The magnitude of these point loads was such that they would produce the same lateral bending moment at midspan as the uniform loading would have. An uplift pressure was applied to the trusses above the lintel, which would tend to bend the lintel upwards and put the laterally loaded bottom chord into compression.

The manufacturer's concern was about the performance of this bottom chord member acting as a laterally loaded column without any significant lateral support. Figure 23 shows the forces on the lintel. The lateral displacement at midspan of the bottom chord was measured to give an indication of the performance of the lintel beam.

The combined loading was applied to the lintel in six equal increments up to design load and the lateral displacement was measured at each increment. At the combined design lateral and uplift loads the lintel had deflected 3.0 mm laterally. This is 1/620 of the span which is well below
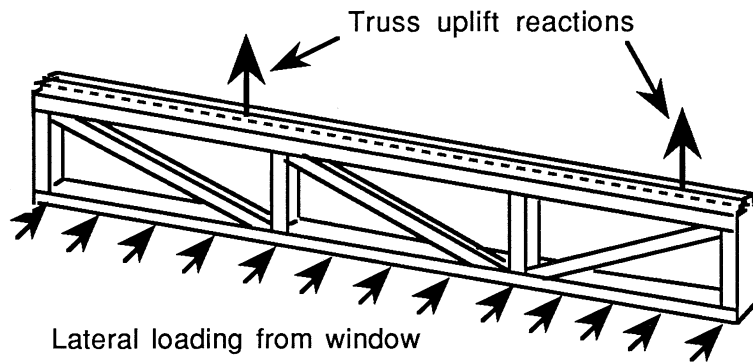
FIGURE 23   Loading on Lintel Beam

normally accepted deflection limits of span/300.   The combined loading was then continued to twice design loads by which time the lateral deflection had increased to 7.7 mm.   At this load there was no evidence of any distress and the lateral deflection was still only 1/240 of the span. After removal of the load the lintel retained a permanent lateral deflection of 1.2 mm.   A graph of lateral deflection of the lintel beam up to twice the combined design loads is given in Figure 24.   The vertical scale is the lateral pressure acting on the sliding door.
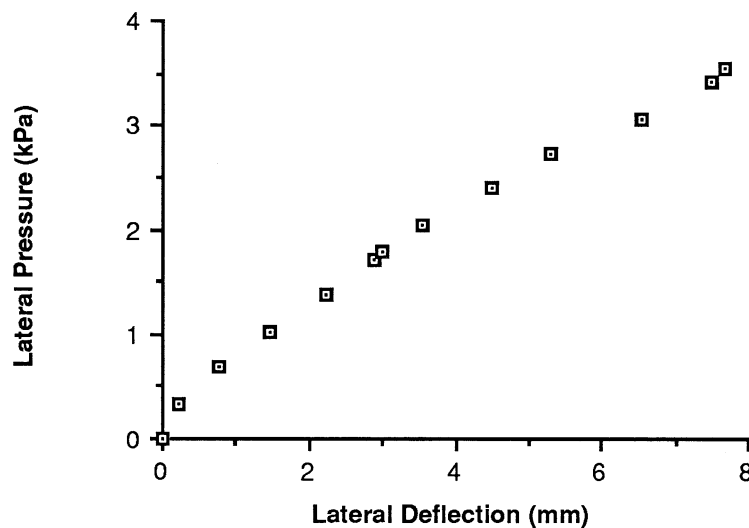
FIGURE 24   Lateral Displacement of Lintel Beam

It was decided that there was no reason to break the lintel beam as it would cause an inconvenience and produce little extra information about its performance.   Loading the lintel to the combination of twice design uplift and lateral loads has shown that there should be no reason for concern about the performance of the member.

8   CONCLUSIONS

The conclusions given in this report relate only to the shape and size of the Nu-Steel house tested. They do not necessarily apply to other shapes and should not be extrapolated to designs for more extreme wind conditions.

The general conclusion that can be drawn from these tests is that the Nu-Steel house performed well when subjected to global pressures that would engulf the whole building. The structure was able to resist the combined lateral and uplift pressures appropriate for terrain category 3. However its performance in resisting local pressures was not so good in that failure of some batten/rafter joints indicates a problem with high local uplift pressures.

Particular conclusions that can be drawn from the test series are as follows:

(a)   The Nu-Steel house resisted the combined racking and uplift pressures calculated for wind blowing perpendicular to the length of a house in terrain category 3 of a tropical cyclone area (classified as Region C in AS 1170.2 - 1989). The only evidence of failure during the 10,200 cycles of load was the partial withdrawal of one of the hold-down bolts securing the bottom plate to the concrete slab. Despite this, the house still resisted the specified overload of two times the design uplift and lateral pressures for that wind direction and therefore satisfied the global test criteria.

(b)   The test highlighted one of the difficulties in securing bottom plates to a concrete slab. Because the plate is located so close to the edge of the slab the manufacturer correctly used an alternative method to the expansion anchor. His instruction diagram is careful to show that the hold-down bolt must be placed with the bend pointing inwards. But the task of accurately installing 63 such bolts in the slab before it cures may be too much for the owner/builder at whom the kit is marketed.

(c)   The results of the tests for wind blowing parallel to the ridge led to some initial concern. Failure of the batten/rafter joints occurred at the static overload value of 1.87 times the actual uplift design pressure. This is not quite sufficient to satisfy the test criteria, which requires a factor of two times the design pressure. However the pressures calculated for test were based on the 1983 edition of the Wind Loading Code, to which the house would have been designed. Pressures calculated from the 1989 edition, which specifies lower wind speeds for tropical cyclone areas, would mean that the static overload value would be 2.07 times the design pressure. On this basis the house would satisfy the criteria for wind

parallel to the ridge.

(d)   Although the total house can be considered to have satisfied the global cyclic loading criteria for wind blowing either parallel or perpendicular to its length, the batten/rafter connection detail still remains questionable. In practice such a joint would have to resist high local uplift pressures approaching the value at which failure occurred. These results indicate that the joint would not have the appropriate margin of safety to cope with those forces.

(e)   The strength of the batten/rafter joints appeared to have been reduced by the the cyclic loading, as the failure load of the joint was well below the anticipated load for such a connection.

(f)   During the static overload for the wind blowing perpendicular to the length of the house, the vertical displacements at the ends of the trusses were very small. With the exception of the those trusses over the wall where the hold-down bolts were withdrawing from the slab, the maximum vertical displacements at design uplift and lateral pressure was only 2 mm. Even at two times design load the maximum vertical displacements that were unaffected by bolt withdrawal were 6 to 7 mm. There was no single obvious cause for the displacement although the bottom plate remained slightly curved after removal of the overload.

(g)   During the overload phase the trusses in the bedroom area lifted at their centre a similar amount to those in the open lounge area, indicating that the internal walls did not provide the trusses with any additional hold-down. The bond of the cornice to the walls would have been broken by then.

(h)   There was very little vertical displacement of the gable ends of the house. As the gable trusses were supported along their length they acted more as extensions of the walls than as trusses.

(i)   As no uplift pressures were applied directly to the roofing, the effect of overdriving the screws could not be determined. However it is the author's opinion that the cracks induced by overdriving would have caused premature failure. In practice such bad workmanship would probably have led to a problem with leakage before the house had to cope with a tropical cyclone.

(j)   The house was very stiff in racking. Preliminary tests during the construction showed that both the roofing and the ceiling were acting as stiff diaphragms, dispersing the applied forces to other elements or to ground.

(k)   The internal wall lining provided all of the bracing necessary to

resist the lateral design pressures, although it was fixed to the framework in the conventional manner and not to bracing wall specifications.

(l)   The cornice played a significant part in stiffening the house as it bound the internal wall lining to the ceiling. Although the roofing acted as a very stiff diaphragm when it was initially installed, once the cornice was in place there was virtually no difference in lateral response of the house with or without the roofing.

(m)   The diagonal cross braces on the internal walls had no measurable effect on the stiffness of those walls. Their only useful function would be in keeping the frame square during transportation and construction.

(n)   With an internal space of approximately 15.6 x 4.6 m the house resisted lateral forces of 2.7 times design load with deflections of only about 2 mm. The internal walls in the kitchen and wet areas were obviously providing the bracing.

(o)   The lintel beams were adequately strong and stiff. A beam over one of the 1.8 m window openings had a lateral deflection of only 3 mm under a combination of design lateral and uplift loading. At twice this loading the deflection was 7.7 mm.

9   ACKNOWLEDGEMENTS

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