TWP Timber and Wood Products
Research Centre

# WALL BRACING TEST RESULTS ON 4.5 MM PLYWOOD SHEATHED, JD5 AND JD6 PINE FRAMED WALL PANELS
for
DINDAS LEW PTY LTD

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TWP Report No. 131 B
April, 1986

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## SUMMARY

Two 2.4 m high x 2.7 m long wall frames consisting of 70 x 35 mm studs at 450 mm centres and 70 x 45 mm top and bottom plates were constructed from F5 pine. One panel was of Spruce Pine Fir with a joint strength group JD5, the other was Hem Fir of joint strength group JD6. Each frame was sheathed on one side only using 4.5 mm thick, F11 D/D structural grade plywood. Sheathing connectors were 2.8 mm diameter x 30 mm long galvanised clouts hand driven at 150 mm centres around edges and 300 mm centres on internal studs. Panels were tested in racking, the plywood was then stripped, the frames reversed and resheathed and the panels retested in racking.

## WALL BRACING TESTS FOR
## DINDAS LEW QLD PTY LTD

### 1. INTRODUCTION

The testing program described herein was performed on 12 April, 1986 for Dindas Lew Qld Pty Ltd on plywood sheathed, timber framed wall panels constructed at Capricornia Institute. The plywood, 2440 x 900 mm x 4.5 mm thick F11, D/D structural grade was supplied by the Plywood Association of Australia Ltd, Brisbane. The timber framing material, 70 x 35 studs and 70 x 45 mm top and bottom plates was F5, Spruce Pine Fir and Hemfir supplied by the client.

Material for two frames, nominally 2.5 m high x 2.7 m long, was provided, one in joint strength group JD5 the other in JD6. Each frame, with studs at 450 mm centres was sheathed one side only and tested in racking. The sheathing was then stripped and new sheathing connected to the opposite side and the panel tested in racking with the load applied to the opposite end to that of the first test.

The nailing pattern used to effect connection between sheathing and timber framing was the standard 150 mm centres around edges and 300 mm centres on internal studs. The nails were 2.8 mm diameter, 30 mm long galvanised clouts.

### 2. PANEL CONSTRUCTION

No special care was taken during fabrication of the panels, construction techniques being representative of normal site practice. Features of note concerning panel construction include:

(i) stud centres were 450 mm for the four panels tested.

(ii) plywood sheathing was fastened to one side only of a panel.

(iii) secondary connection between top and bottom plates and studs was effected by a single 3.8 mm diameter x 100 mm long jolt head nail hand driven into a predrilled hole.

(iv) primary connection between plywood sheathing, top and bottom plates, and studs was obtained by means of hand driven 30 mm long x 2.8 mm diameter galvanised clouts.

(v) plywood sheets were connected within 2 mm of the bottom edge of the bottom plate with clouts driven 10 mm from edges.

Figure 1 shows the wall specifications for the panels tested.

### 3. ALLOWABLE RACKING LOAD

For a wall panel to be deemed adequate as a structural component capable of resisting applied racking loads it must be:

(a) stiff enough to resist the design loads without excessive racking deflection at eaves level.

(b) strong enough to resist the design loads and still provide an adequate safety margin on its ultimate load carrying capacity be this either connector or material dependent.

## WALL PANEL SPECIFICATIONS

**Test Panel No:** DL N^os 1 & 2 (JD5)
3 & 4 (JD6)

**Test Load Type:** Racking ~~Uplift~~

**Date of Test:** ____________

### TIMBER FRAMING

| | Studs | Top & Bottom Plates | Rafters/Joists |
|---|---|---|---|
| Size | 70 x 35 | 70 x 45 | x |
| Grade | F 5 | F 5 | F |
| m.c. | Av. 10 % | Av. 10 % | % |
| Spacing (S) | 450 mm | 2.7 m. | |
| Density | JD5: 417 kg/m³<br>JD6: 362 kg/m³ | | |

**SHEATHING:** Plywood

**Size:** 2440 x 900 ; **Type:** F11, D/D ; **Thick:** 4 mm

### FASTENERS

**Type:** 2.8 mm diameter x mm galvanised clouts

**Spacing:** Top Plate 150 ; Bot. Plate 150 ; Int. Studs 300 ; Edge Studs 150

### CYCLONE BOLT

**Fitted:** No ; **Diameter:** —

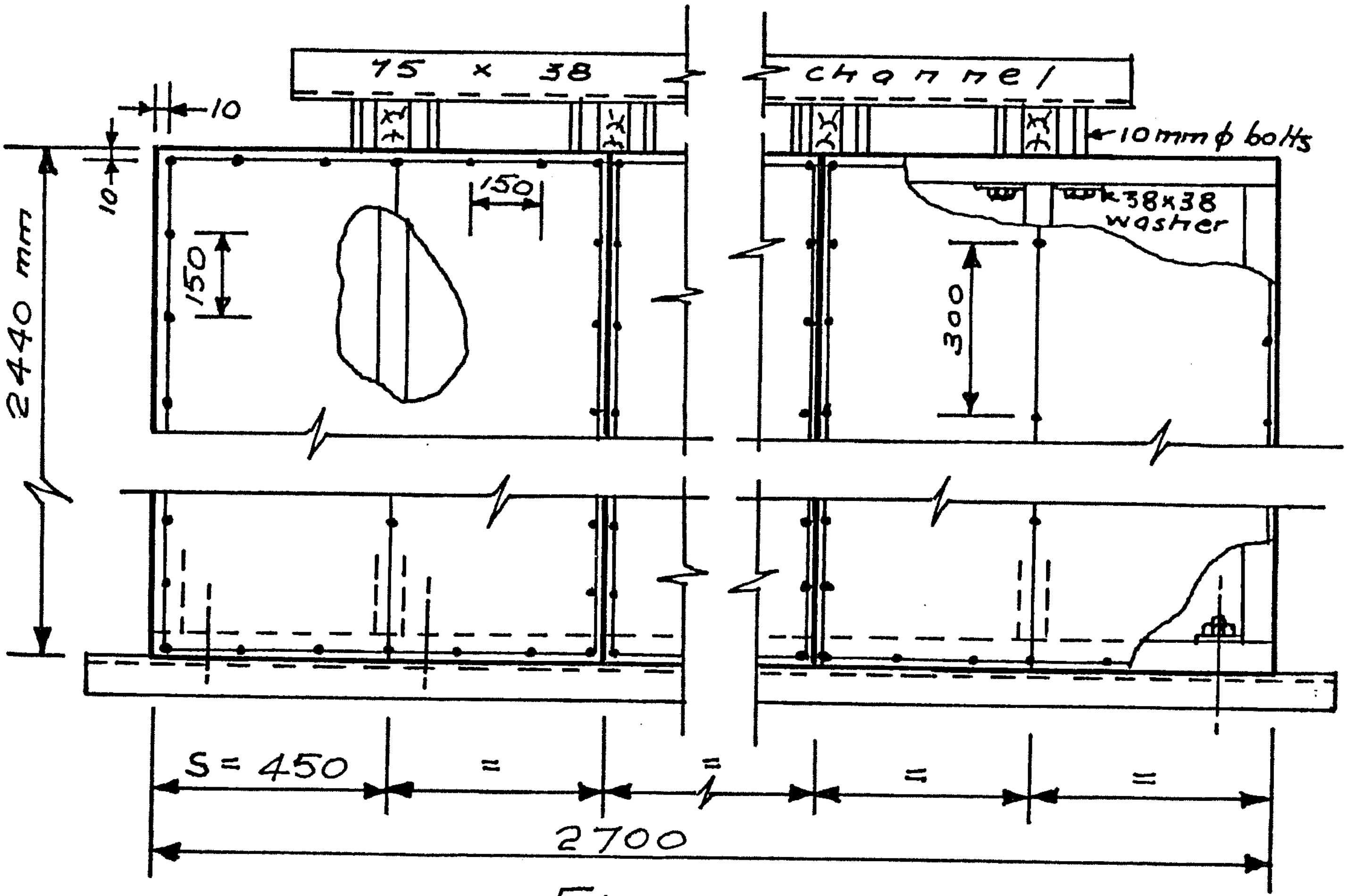



FIGURE 1

(c) remain stable, ie, show no significant signs of buckling to the design load.

Stiffness criteria for these tests were based on the racking deflection being less than the panel height/300, ie, less than 8 mm.

Based on a design wind speed of 33 m/s, modification of the allowable racking load/metre of 2.25 kN for standard bracing wall construction (1) for 42 m/s wind gives:

$$R_D = \left(\frac{33}{42}\right)^2 . \ 2.25 . L_W \quad \text{kN}$$

where $R_D$ = total design racking load

$L_W$ = length of bracing wall

Hence:

$R_D$ = 3.75 kN for a 2.7 m wall length.

Since only two panels were tested for each joint strength groups (2) suggests:

$$\text{Design Racking Load/metre} = \frac{\text{Average Panel Failure Load}}{\text{Load Factor x 2.7}} \qquad (1)$$

where:

Load Factor = 2.2
Panel Length = 2.7 m

## 4. LOADING RIG

### 4.1 General

Loading of all panels was done in the end portal frame of the Three Dimensional Loading Frame located in the Heavy Structures Laboratory, Capricornia Institute. Figure 2 shows diagrammatically a typical panel located in the loading frame prior to testing in either racking or uplift.

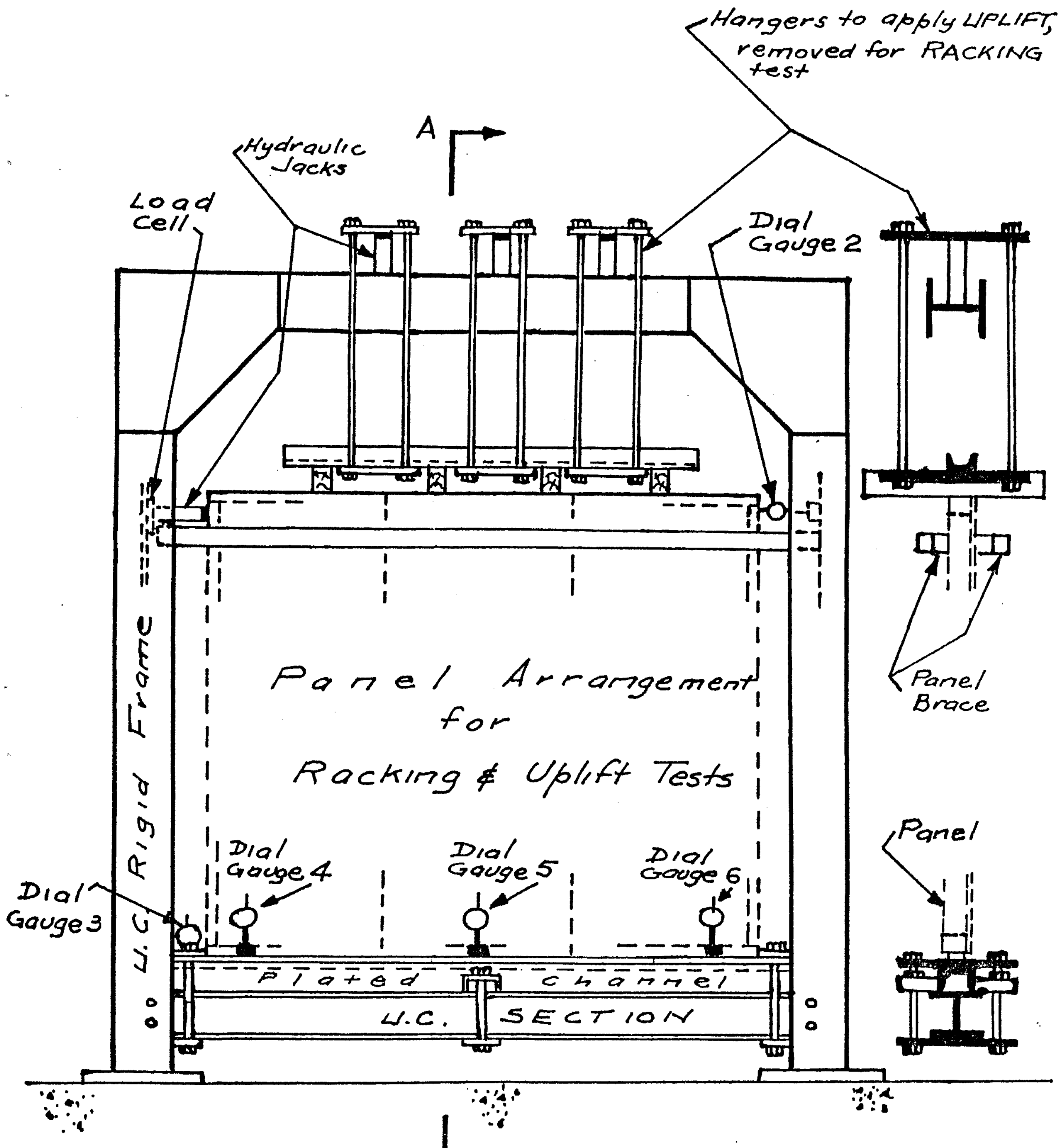
Racking loads were applied by means of 120 kN Ritch hydraulic jack reacting against the rigid portal column through a 50 kN load cell. Activation of the jack was effected through a hand operated pump and the load monitored by a digital voltmetre connected to the load cell.

### 4.2 Racking Test Arrangement

The bottom plate of the test panel was bolted to the plated channel as shown in Figure 1. Such a set-up simulates slab-on-ground construction and eliminates extraneous panel rotations thus providing a direct means for estimating panel strength and stiffness. Figure 2 shows the method of clamping the plated channel to a UC beam located within the end portal.

75 x 50 mm x F14 timber members positioned near the top of the panel, one either side, prevent the top plate from buckling laterally.

Dial gauge 2, shown to be attached to the portal, was in fact located on a reinforced concrete beam of the Structures Laboratory thus allowing the absolute racking deflection




ELEVATION

SECTION A-A

TEST RIG ARRANGEMENT

FIGURE 2

to be monitored directly. Dial gauge 3 measures any rigid body movement inherited by the panel and dial gauges 4, 5, and 6 monitor vertical panel movement providing necessary information to establish the centre of rotation, if required. With the set-up described herein there is no need to estimate the centre of rotation.

## 5. TEST PROCEDURE

All panels were preloaded to a total racking load of 2.25 kN, during which no deflection readings were taken. All dial gauges were then zeroed.

Panels were then reloaded in increments of 0.45 kN to the estimated design load fo 3.75 kN. During this loading all dial gauge readings were recorded. The load was held for five minutes, dial gauge readings taken to record the creep, the load then being released. After a further five minute recovery period the reading of dial gauge 2 was taken whilst dial gauges 3, 4, 5 and 6 were rezeroed.

Panels were then loaded to failure in 0.9 kN increments and all dial gauge readings recorded. This procedure was continued until excessive creep precluded reading of all gauges except 2, the one monitoring racking deflection.

During loading panel response was closely observed. In particular, visual inspections were done to check that buckling of the sheathing did not occur prematurely, ie, at loads decidedly less than the estimated design racking load.

The test procedure followed as closely as practicable that described in (3) for prototype testing.

## 6. TEST RESULTS

From the reduced data obtained during testing load-deflection plots were prepared for each panel. A brief discussion of these results follows.

### 6.1 TEST PANEL DL 1

Timber framing used to construct the panel was 70 x 35 mm studs and 70 x 45 mm top and bottom plates of F5 pine from material of joint strength group JD 5. After fabrication the panel was light and easily handled manually. Followng location in the loading frame the panel was flat and free of construction buckles.

Time-load-deflection results are given in Table 1 for the system loaded to the design and failure loads respectively. Figure 3 shows a load-deflection plot of the data contained in Table 1. The load to failure curve is linear to a load of about 1.8 kN and bilinear to the design load of 3.75 kN. The deflection at the design load is approximately 2.6 mm which includes about 0.5 mm of permanent deformation from the initial loading.

During loading to failure the first audible sounds were emitted at a load of 4.5 kN. Creep was evidenced at a load of 5.4 kN, being accompanied by visible signs of buckling, and sounds of sheathing tearing.

Panel failure occurred at a load of 10.8 kN. Failure was gentle resulting in plywood tear-out behind the first five nails in the bottom plate at the loaded end. Plate 1 shows the failure mode together with the amount of stud/bottom plate separation at the loaded end. Plate 2 shows the nail deformation pattern and the tendency of the middle sheet and the one nearest the loaded end to work as one, ie, there is little relative rotation between them. This response is typical of panels requiring the sheathing to resist both the racking and overturning forces.

There was no visible evidence of plywood buckling up to the design load.

# WALL PANEL TEST RESULTS

TEST PANEL NO. DL 1

TEST LOAD TYPE: RACKING

| Time (min.) | Racking Load (kN) | Dial Gauge Reading (mm) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Test Frame | Panel Defln. | Rigid Body | Horiz. Defln. | Panel Rotation | | | Actual Racking |
| | | 1 | 2 | 3 | Δ | 4 | 5 | 6 | $\Delta_R$ |
| 0:00 | 0·00 | | 0·00 | 0·00 | | 0·00 | 0·00 | 0·00 | 0·00 |
| | 0·45 | | 0·19 | 0·00 | | 0·00 | 0·00 | 0·00 | 0·19 |
| | 0·90 | | 0·37 | 0·01 | | 0·01 | 0·00 | 0·00 | 0·36 |
| | 1·35 | | 0·59 | 0·01 | | 0·02 | 0·00 | 0·00 | 0·58 |
| | 1·80 | | 0·82 | 0·01 | | 0·05 | 0·00 | 0·00 | 0·81 |
| | 2·25 | | 1·08 | 0·01 | | 0·09 | 0·01 | 0·01 | 1·07 |
| | 2·70 | | 1·42 | 0·01 | | 0·18 | 0·01 | 0·01 | 1·41 |
| | 3·15 | | 1·81 | 0·01 | | 0·26 | 0·01 | 0·02 | 1·80 |
| | 3·60 | | 2·23 | 0·01 | | 0·39 | 0·01 | 0·02 | 2·22 |
| 4:50 | 3·75 | | 2·38 | 0·01 | | 0·43 | 0·01 | 0·02 | 2·37 |
| 10:00 | 3·75 | | 2·51 | 0·02 | | 0·48 | 0·01 | 0·03 | 2·49 |
| 11:20 | 0·00 | | 0·57 | 0·01 | | 0·28 | 0·01 | 0·00 | 0·56 |
| | | | | | | | | | |
| 16:20 | 0·00 | | 0·48 | 0·00 | | 0·00 | 0·00 | 0·00 | 0·48 |
| | 0·90 | | 0·84 | 0·00 | | 0·00 | 0·00 | 0·00 | 0·84 |
| | 1·80 | | 1·28 | 0·01 | | 0·02 | 0·00 | 0·00 | 1·27 |
| | 2·70 | | 1·81 | 0·01 | | 0·10 | 0·00 | 0·01 | 1·80 |
| | 3·60 | | 2·36 | 0·01 | | 0·21 | 0·00 | 0·04 | 2·35 |
| | 4·50 | | 3·21 | 0·01 | | 0·50 | 0·00 | 0·06 | 3·20 |
| | 5·40 | | 4·55 | 0·02 | | 0·96 | 0·09 | 0·06 | 4·53 |
| | 6·30 | | 5·95 | 0·02 | | 1·45 | 0·28 | 0·04 | 5·93 |
| | 7·20 | | 7·75 | 0·03 | | 2·35 | 0·61 | 0·09 | 7·72 |
| | 8·10 | | 9·85 | 0·04 | | 3·45 | | | 9·81 |
| | 9·00 | | | | | | | | |
| | 9·90 | | | | | | | | |
| 24:00 | 10·80 | | 22+ | | | | | | 22+ |

TABLE 1

PROOF LOAD: 3·75 kN

DEFLECTION: 2·37 mm

ULTIMATE LOAD: 10·8 kN

DEFLECTION: + 22 mm

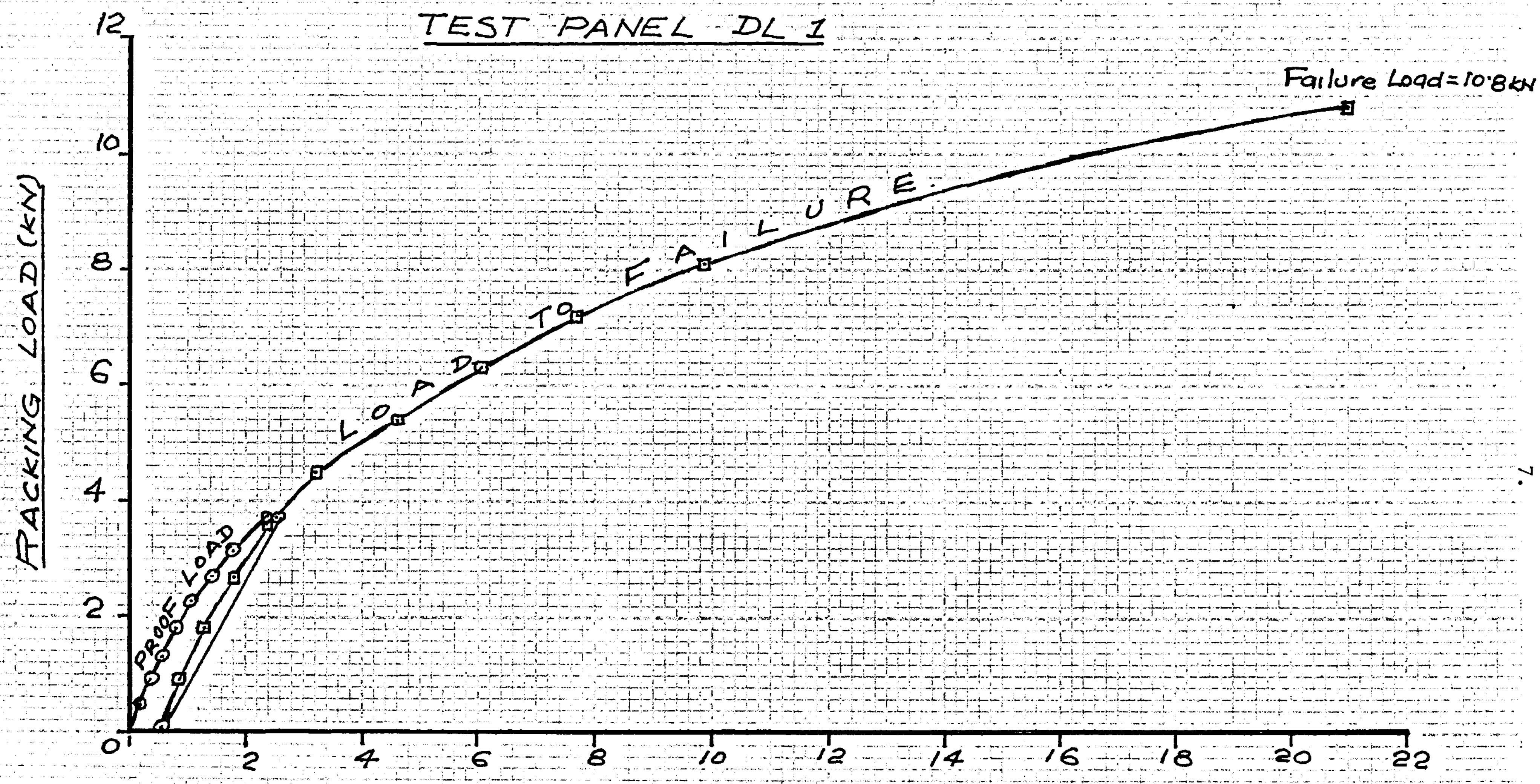
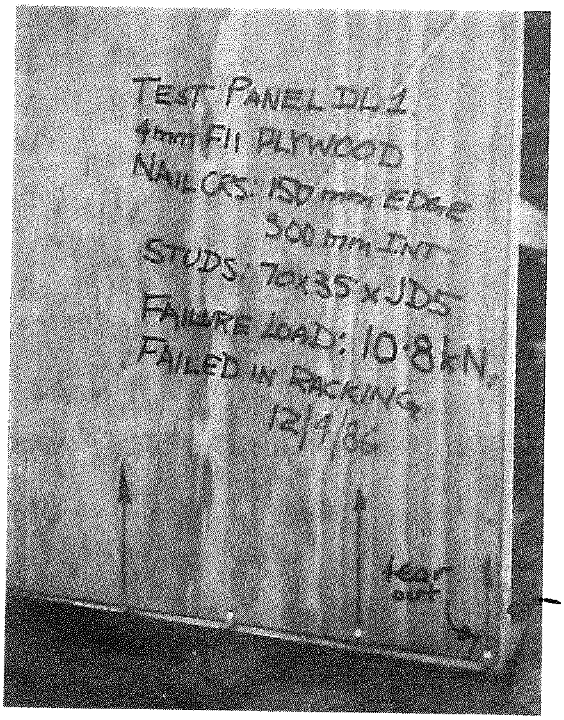
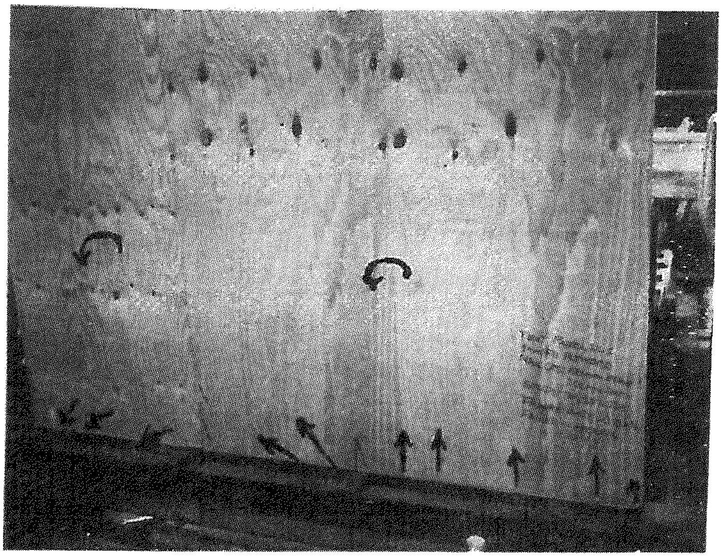

TEST PANEL DL 1
Failure Load=10·8kN
PROOF LOAD
LOAD TO FAILURE
RACKING LOAD (kN)
RACKING DEFLECTION (mm)
0
2
4
6
8
10
12
14
16
18
20
22

FIGURE 3




stud/bottom plate separation.

FAILURE MODE
PLATE 1



DIRECTION OF NAIL MOVEMENTS
PLATE 2

## 6.2 TEST PANEL DL 2

To construct the panel the plywood sheathing was stripped from test panel DL 1. The frame was turned over, squared up and resheathed with new plywood such that the racking load was applied to the opposite corner to that for DL 1.

Time-load-deflection results are given in Table 2 for the system loaded to the design and failure loads respectively. Figure 4 shows a load-deflection plot of the data contained in Table 2. The load to failure curve is reasonably linear to the design load of 3.75 kN. The deflection at the design load is less than 2.5 mm including about 0.4 mm of permanent deformation from the initial loading.

During load to failure the first audible sounds were emitted at a load of 2.7 kN.

Panel failure occurred at a load of 11.4 kN. Failure resulted in tear-out of the plywood behind the first 9 nails in the bottom plate at the loaded end as shown in Plate 3. There was also some indication of a tendency towards nail withdrawal in some of the nails in the bottom plate nearest the loaded end. The tendency of the first two sheets nearest the loaded end to work as one was again in evidence.

There were no visible signs of plywood buckling up to the design load.

## WALL PANEL TEST RESULTS

TEST PANEL NO. DL2 TEST LOAD TYPE: RACKING

| Time (min.) | Racking Load (kN) | Dial Gauge Reading (mm) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Test Frame | Panel Defln. | Rigid Body | Horiz. Defln. | Panel Rotation | | | Actual Racking |
| | | 1 | 2 | 3 | Δ | 4 | 5 | 6 | $\Delta_R$ |
| 0:00 | 0.00 | | 0.00 | 0.00 | | 0.00 | 0.00 | 0.00 | 0.00 |
| | 0.45 | | 0.18 | 0.01 | | 0.00 | 0.00 | 0.00 | 0.17 |
| | 0.90 | | 0.34 | 0.01 | | 0.00 | 0.00 | 0.00 | 0.33 |
| | 1.35 | | 0.52 | 0.01 | | 0.02 | 0.00 | 0.00 | 0.51 |
| | 1.80 | | 0.73 | 0.01 | | 0.05 | 0.00 | 0.00 | 0.72 |
| | 2.25 | | 1.00 | 0.01 | | 0.09 | 0.00 | 0.00 | 0.99 |
| | 2.70 | | 1.28 | 0.01 | | 0.15 | 0.00 | 0.00 | 1.27 |
| | 3.15 | | 1.63 | 0.01 | | 0.23 | 0.00 | 0.00 | 1.62 |
| | 3.60 | | 1.98 | 0.01 | | 0.30 | 0.04 | 0.00 | 1.97 |
| 3:30 | 3.75 | | 2.13 | 0.01 | | 0.35 | 0.06 | 0.00 | 2.12 |
| 9:30 | 3.75 | | 2.28 | 0.01 | | 0.39 | 0.08 | 0.00 | 2.27 |
| 10:00 | 0.00 | | 0.46 | 0.01 | | 1.14 | 0.08 | 0.00 | 0.45 |
| | | | | | | | | | |
| 15:00 | 0.00 | | 0.39 | 0.00 | | 0.00 | 0.00 | 0.00 | 0.39 |
| | 0.90 | | 0.73 | 0.01 | | 0.00 | 0.00 | 0.00 | 0.72 |
| | 1.80 | | 1.12 | 0.01 | | 0.04 | 0.00 | 0.00 | 1.11 |
| | 2.70 | | 1.62 | 0.01 | | 0.13 | 0.00 | 0.00 | 1.61 |
| | 3.60 | | 2.11 | 0.01 | | 0.22 | 0.00 | 0.00 | 2.10 |
| | 4.50 | | 2.86 | 0.01 | | 0.41 | 0.08 | 0.00 | 2.85 |
| | 5.40 | | 3.79 | 0.01 | | 0.74 | 0.22 | 0.02 | 3.78 |
| | 6.30 | | 5.04 | 0.01 | | 1.19 | 0.42 | 0.10 | 5.03 |
| | 7.20 | | 6.65 | 0.02 | | 1.89 | 0.78 | 0.25 | 6.63 |
| | 8.10 | | 8.50 | 0.02 | | 2.81 | 1.20 | 0.50 | 8.48 |
| | 9.00 | | | | | | | | |
| | 9.90 | | | | | | | | |
| | 10.80 | | | | | | | | 21+ |
| 20:00 | 11.40 | | 21+ | | | | | | |

TABLE 2

PROOF LOAD: 3.75 kN DEFLECTION: 2.13 mm

ULTIMATE LOAD: 11.4 kN DEFLECTION: +21 mm

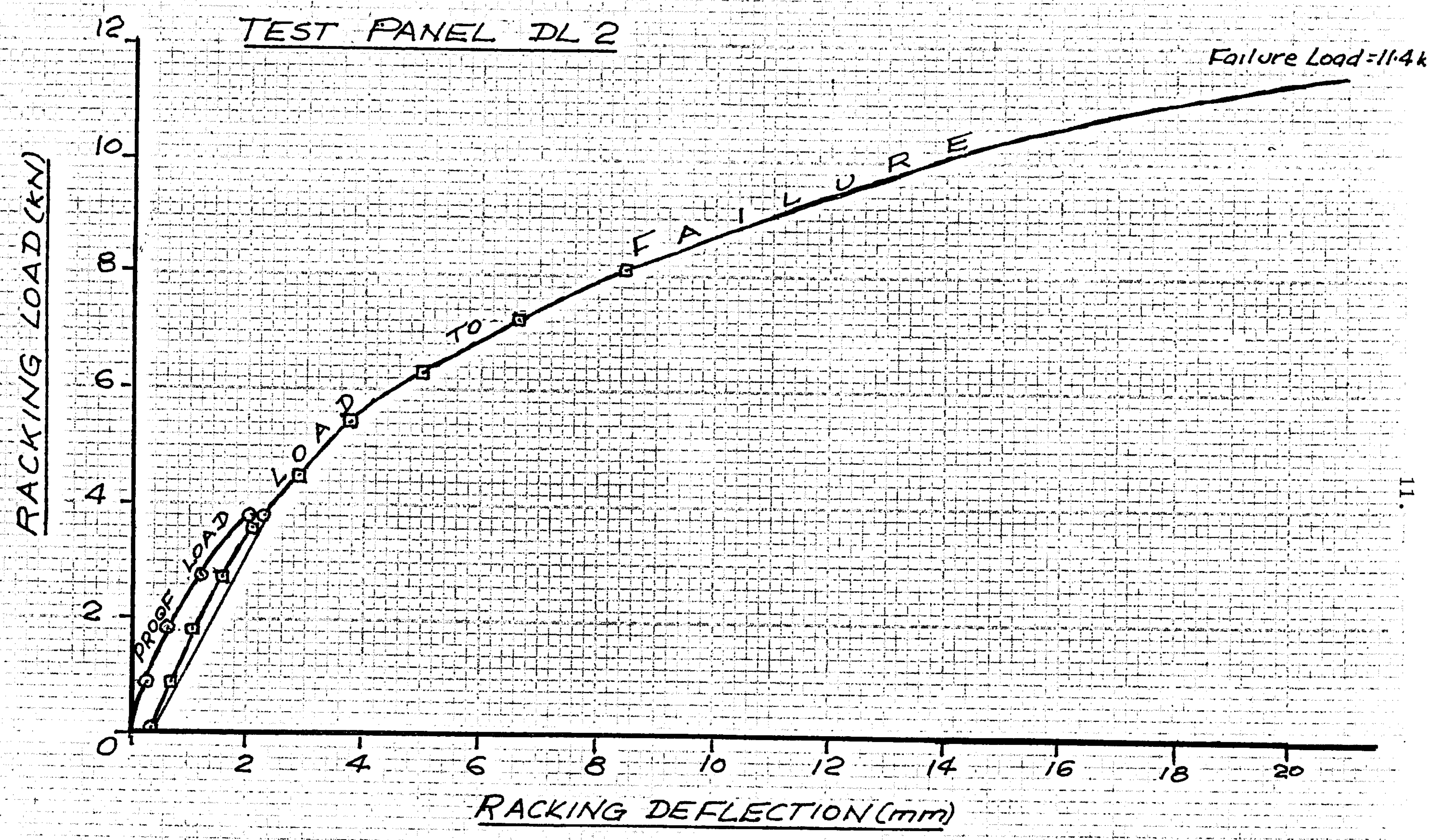

TEST PANEL DL 2
Failure Load = 11.4 k
PROOF LOAD
LOAD TO FAILURE
RACKING LOAD (kN)
RACKING DEFLECTION (mm)
0
2
4
6
8
10
12
14
16
18
20

FIGURE 4

## 6.3 TEST PANEL DL 3

Timber framing used to construct the panel was 70 x 35 mm studs and 70 x 45 mm top and bottom plates of F5 pine from material of joint strength group JD 6. The panel was easy to handle and flat when located in the loading frame.

Time-load-deflection results are given in Table 3 for the system loaded to the design and failure loads respectively. Figure 5 shows a load-deflection plot of Table 3 data. The load to failure curve is at no stage linear up to the design load. The deflection at the design load is in excess of 4 mm but includes 1.2 mm of permanent deformation from the initial loading.

During initial loading to the design value sounds of tearing were audible at a load of 3.6 kN. Considerable separation of the first stud from the loaded end and the bottom plate was also observed.

Panel failure occurred at a load of 8.6 kN. Failure resulted in tear-out of the plywood behind the second, fifth, and sixth nails in the bottom plate at the loaded end. The first nail in the bottom plate at the loaded end (see Plate 4) had withdrawn significantly and was standing some 12 mm clear of the plywood. There was little relative rotation between the first two sheets of plywood from the loaded end.

There were no visible signs of plywood buckling up to the design load. The first signs of buckling became apparent at a load of 5.4 kN.

# WALL PANEL TEST RESULTS

TEST PANEL NO. DL 3 TEST LOAD TYPE: RACKING

| Time (min.) | Racking Load (kN) | Dial Gauge Reading (mm) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Test Frame | Panel Defln. | Rigid Body | Horiz. Defln. | Panel Rotation | | | Actual Racking |
| | | 1 | 2 | 3 | Δ | 4 | 5 | 6 | ΔR |
| 0:00 | 0.00 | | 0.00 | 0.00 | | 0.00 | 0.00 | 0.00 | 0.00 |
| | 0.45 | | 0.20 | 0.00 | | 0.01 | 0.00 | 0.00 | 0.20 |
| | 0.90 | | 0.39 | 0.00 | | 0.05 | 0.00 | 0.00 | 0.39 |
| | 1.35 | | 0.67 | 0.00 | | 0.11 | 0.00 | 0.00 | 0.67 |
| | 1.80 | | 1.03 | 0.00 | | 0.19 | 0.00 | 0.00 | 1.03 |
| | 2.25 | | 1.40 | 0.00 | | 0.30 | 0.01 | 0.00 | 1.40 |
| | 2.70 | | 2.05 | 0.00 | | 0.53 | 0.08 | 0.03 | 2.05 |
| | 3.15 | | 2.68 | 0.00 | | 0.76 | 0.17 | 0.03 | 2.68 |
| | 3.60 | | 3.38 | 0.00 | | 1.02 | 0.27 | 0.04 | 3.38 |
| 4:00 | 3.75 | | 3.64 | 0.01 | | 1.14 | 0.31 | 0.04 | 3.63 |
| 9:00 | 3.75 | | 3.84 | 0.01 | | 1.23 | 0.37 | 0.04 | 3.83 |
| 11:00 | 0.00 | | 1.40 | 0.01 | | 0.64 | 0.32 | 0.03 | 1.39 |
| | | | | | | | | | |
| 16:00 | 0.00 | | 1.20 | 0.00 | | 0.00 | 0.00 | 0.00 | 1.20 |
| | 0.90 | | 1.74 | 0.00 | | 0.05 | 0.00 | 0.00 | 1.74 |
| | 1.80 | | 2.35 | 0.00 | | 0.18 | 0.00 | 0.00 | 2.35 |
| | 2.70 | | 3.08 | 0.00 | | 0.37 | 0.00 | 0.00 | 3.08 |
| | 3.60 | | 3.86 | 0.00 | | 0.61 | 0.05 | 0.00 | 3.86 |
| | 4.50 | | 6.03 | 0.01 | | 1.07 | 0.25 | 0.00 | 6.02 |
| | 5.40 | | 6.90 | 0.01 | | 2.00 | 0.70 | 0.04 | 6.89 |
| | 6.30 | | 9.20 | 0.02 | | 3.13 | 1.27 | 0.22 | 9.18 |
| | 7.20 | | | | | | | | |
| | 8.10 | | | | | | | | |
| 22:00 | 8.60 | | +22 | | | | | | 22+ |

TABLE 3

PROOF LOAD: 3.75 kN DEFLECTION: 3.64 mm

ULTIMATE LOAD: 8.6 kN DEFLECTION: +22 mm

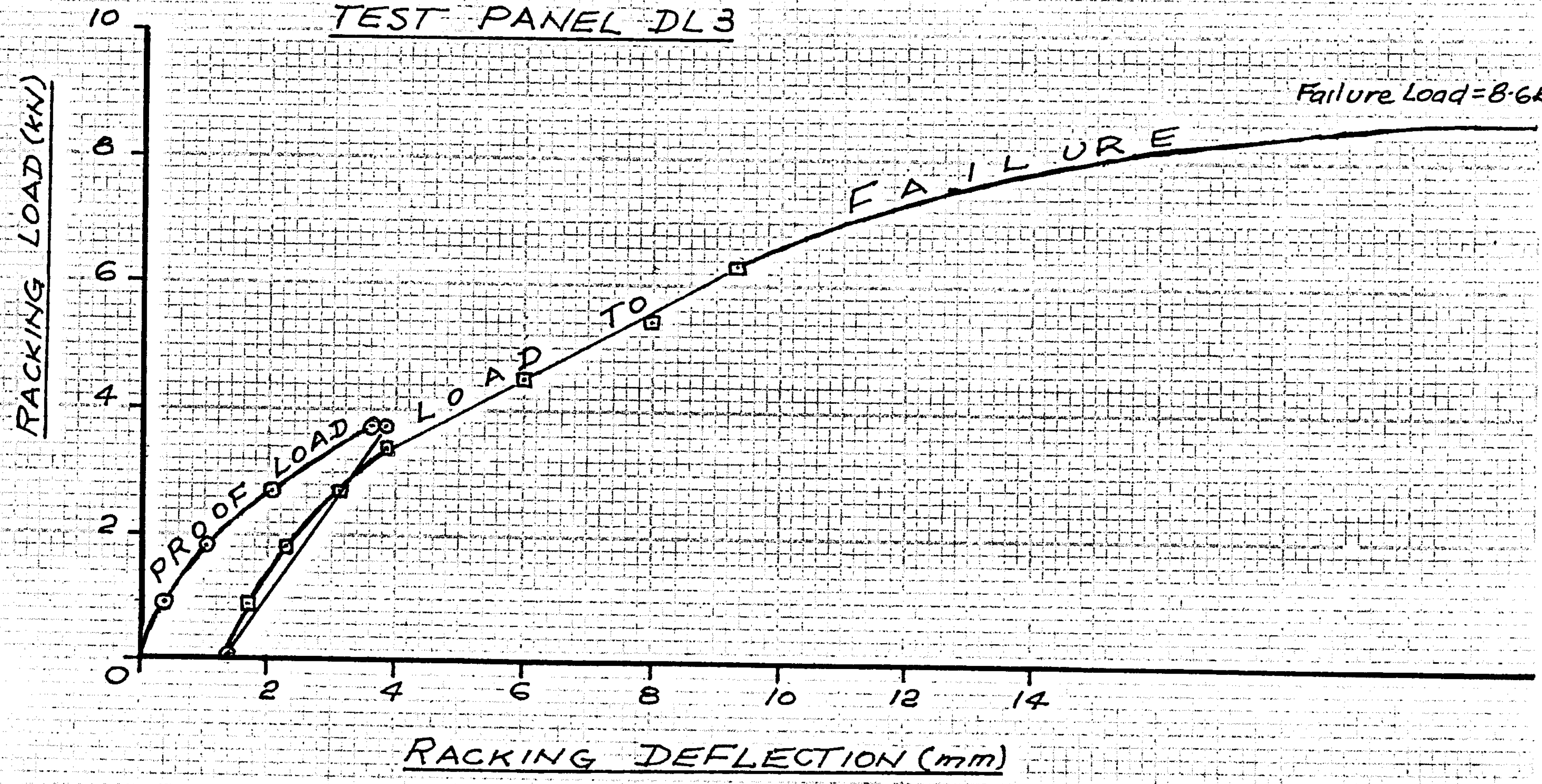

TEST PANEL DL3
Failure Load = 8·6kN.
PROOF LOAD
LOAD TO FAILURE
RACKING LOAD (kN)
RACKING DEFLECTION (mm)
0
2
4
6
8
10
12
14

FIGURE 5

## 6.4 TEST PANEL DL 4

To construct the panel the plywood sheathing was stripped from test panel DL 3. The frame was turned over, squared up and resheathed with new plywood such that the racking load was applied from the opposite corner to that for DL 3.

Time-load-deflection results are given in Table 4 for the system loaded to the design and failure loads respectively. Figure 6 shows a load-deflection plot of Table 4 data. The load to failure curve is linear to the design load of 3.75 kN. The deflection at the design load is somewhat less than 4.5 mm including almost 2 mm of permanent deformation from the initial loading.

During proof loading the first audible sounds were heard at a load of 2.7 kN.

Panel failure occurred at a load of 8.6 kN. Failure resulted in tear-out of the plywood behind the first three nails in the bottom plate at the loaded end as shown in Plate 5. The next three nails were partially popped out of the bottom plate. Tear-out of plywood behind the first four nails from the loaded end of the centre sheet also occurred. Sheathing rotation was similar to that of the other three panels.

There was no visible signs of plywood buckling up to the design load.

# WALL PANEL TEST RESULTS

TEST PANEL NO. DL 4 TEST LOAD TYPE: RACKING

| Time (min.) | Racking Load (kN) | Dial Gauge Reading (mm) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Test Frame | Panel Defln. | Rigid Body | Horiz. Defln. | Panel Rotation | | | Actual Racking |
| | | 1 | 2 | 3 | Δ | 4 | 5 | 6 | $\Delta_R$ |
| 0:00 | 0·00 | | 0·00 | 0·00 | | 0·00 | 0·00 | 0·00 | 0·00 |
| | 0·45 | | 0·22 | 0·00 | | 0·00 | 0·00 | 0·00 | 0·22 |
| | 0·90 | | 0·41 | 0·01 | | 0·02 | 0·00 | 0·00 | 0·40 |
| | 1·35 | | 0·68 | 0·01 | | 0·07 | 0·00 | 0·02 | 0·67 |
| | 1·80 | | 1·06 | 0·01 | | 0·13 | 0·00 | 0·06 | 1·05 |
| | 2·25 | | 1·55 | 0·01 | | 0·24 | 0·00 | 0·10 | 1·54 |
| | 2·70 | | 2·35 | 0·01 | | 0·48 | 0·01 | 0·19 | 2·34 |
| | 3·15 | | 3·00 | 0·01 | | 0·74 | 0·05 | 0·20 | 1·99 |
| | 3·60 | | 3·72 | 0·01 | | 1·00 | 0·12 | 0·20 | 3·71 |
| 4:15 | 3·75 | | 4·04 | 0·01 | | 1·13 | 0·17 | 0·20 | 4·03 |
| 9:45 | 3·75 | | 4·42 | 0·01 | | 1·35 | 0·25 | 0·20 | 4·41 |
| 10:40 | 0·00 | | 2·05 | 0·00 | | 0·97 | 0·25 | 0·19 | 2·05 |
| | | | | | | | | | |
| 16:00 | 0·00 | | 1·89 | 0·00 | | 0·00 | 0·00 | 0·00 | 1·89 |
| | 0·90 | | 2·35 | 0·01 | | 0·02 | 0·00 | 0·00 | 2·34 |
| | 1·80 | | 2·86 | 0·01 | | 0·09 | 0·00 | 0·00 | 2·85 |
| | 2·70 | | 3·54 | 0·01 | | 0·24 | 0·00 | 0·00 | 3·53 |
| | 3·60 | | 4·21 | 0·01 | | 0·41 | 0·00 | 0·01 | 4·20 |
| | 4·50 | | 5·31 | 0·02 | | 0·85 | 0·15 | 0·01 | 5·29 |
| | 5·40 | | 7·05 | 0·02 | | 1·67 | 0·48 | 0·04 | 7·03 |
| | 6·30 | | 9·35 | 0·03 | | 2·85 | 1·00 | 0·04 | 9·32 |
| | 7·20 | | | | | | | | |
| | 8·10 | | | | | | | | |
| 20:00 | 8·60 | | +22 | | | | | | +22 |

TABLE 4

PROOF LOAD: 3·75 kN DEFLECTION: 4·04 mm

ULTIMATE LOAD: 8·6 kN DEFLECTION: +22 mm

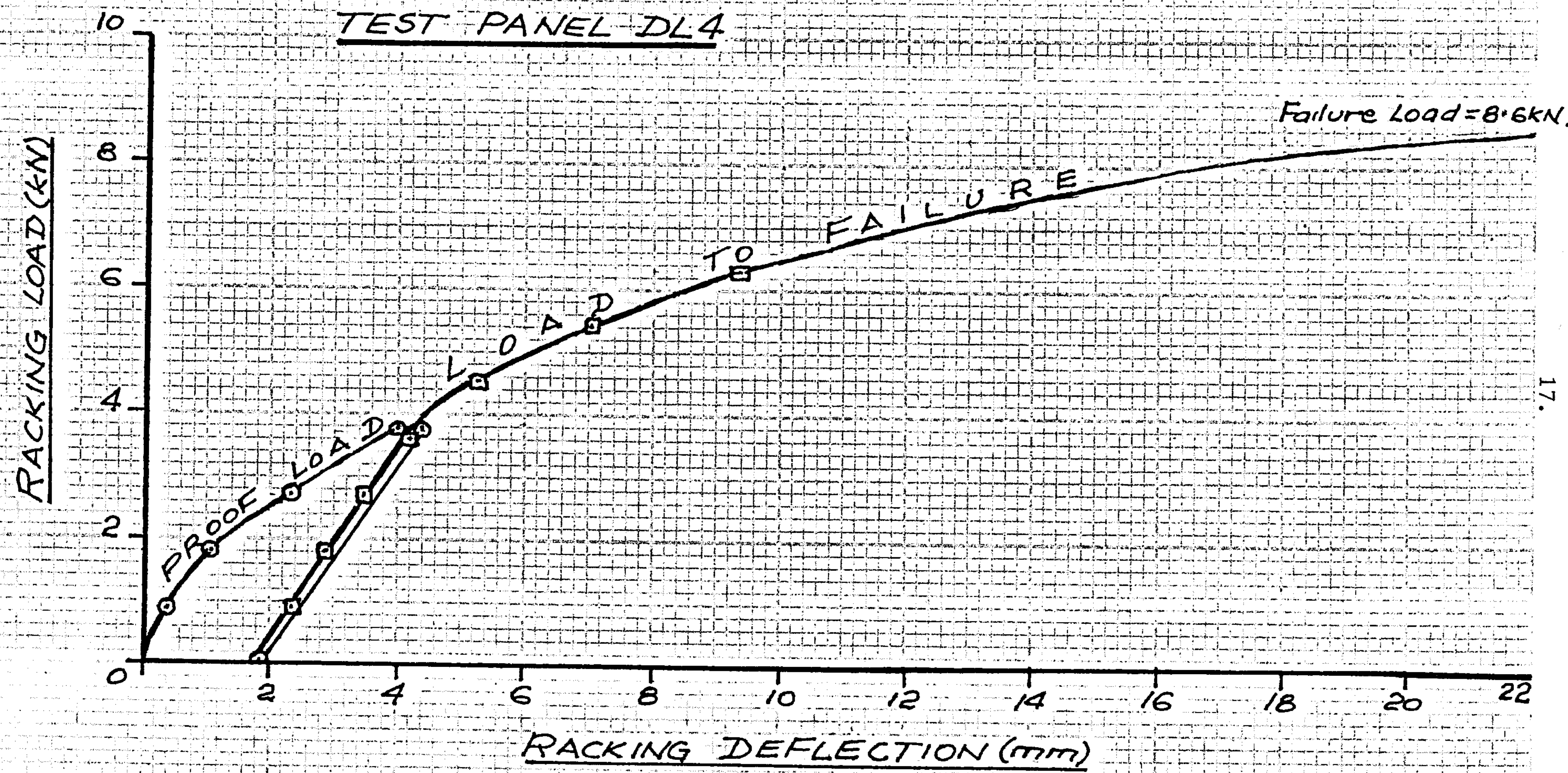

TEST PANEL DL4
Failure Load = 8·6kN.
PROOF LOAD
LOAD TO FAILURE
RACKING LOAD (kN)
0
2
4
6
8
10
RACKING DEFLECTION (mm)
0
2
4
6
8
10
12
14
16
18
20
22

FIGURE 6

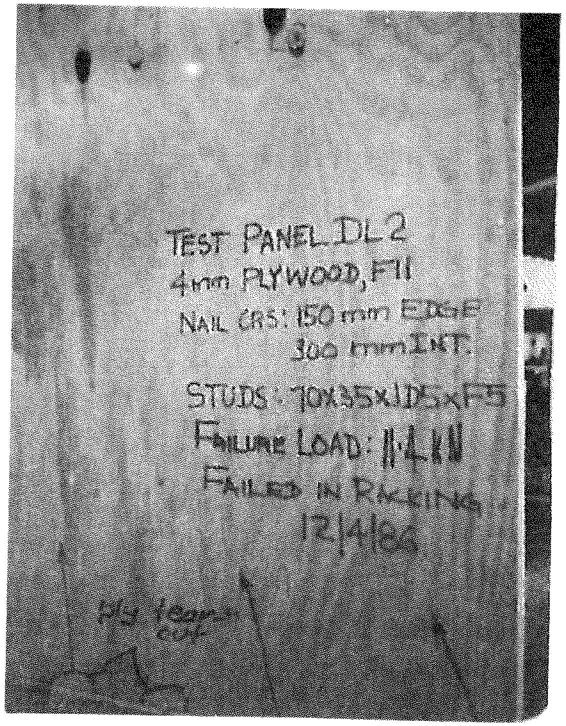



PLATE 3

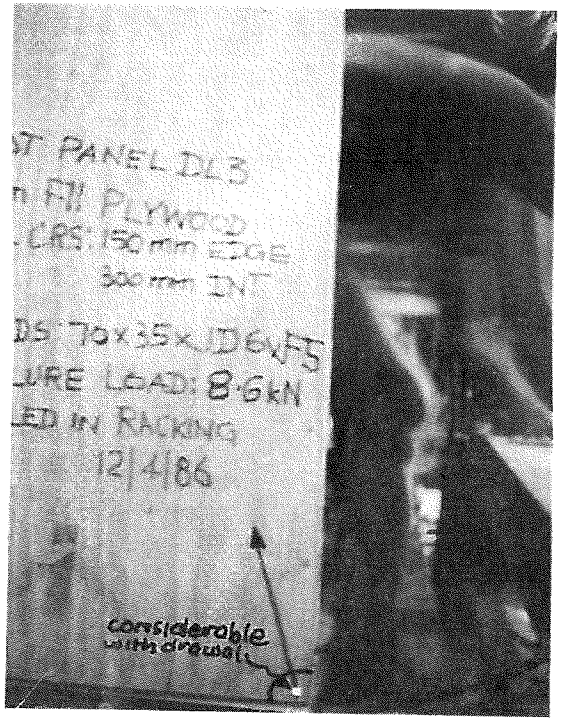



PLATE 4

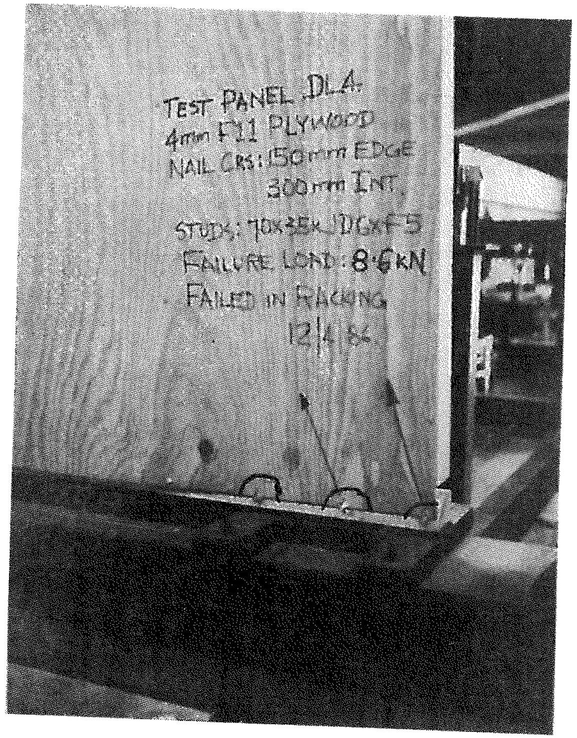



PLATE 5

## 7. OBSERVATIONS & CONCLUSIONS

Overviewing the test program the following observations are worthy of note.

- the 2.4 m long x 2.7 m high panels were light weight and could easily be handled by two mean.
- there was no evidence of plywood buckling up to the estimated design load (edl).
- panel stiffness at the edl was in every case < L/300, ie, 8 mm, and in fact for the worst case (DL3) was half this value.
- JD6 framing results in considerably larger permanent deformations than JD5, eg, from > 1 mm for JD6 and < 0.5 mm for JD5.
- using JD6 framing resulted in certain nails withdrawing substantially at failure.
- tests on the JD5 frame resulted in an average failure load of (10.8 + 11.4)/2 = 11.1 kN. For the JD6 frame the average failure load was 8.6 kN.
- each of the four panels responded such that there was little relative rotation between the two plywood sheets nearest the loaded end. This is a typical response when the high tensile force at the loaded end resulting from overturning tendencies have to be transferred by the nails and sheathing in this vicinity. From the test set-up it would be expected that overturning would be significantly worse than in an actual dwelling. Inclusion of a cyclone rod results in it performing this function thus allowing the sheating and nails to transfer only the shear due to the applied racking load.
- with the exception of DL3 load response was fairly linear, at least to the edl.
- since an edge distance for the nails of 10 mm was used in construction of all panels, it could well be expected that a reduction in racking resistance would result if the minimum distance of 7 mm (1) had been used.

In the opinion of the writer the tested panels did perform satisfactorily as bracing walls. The large amount of permanent deformation associated with loading the JD6 framed panels to the design load is of some concern. However, in view of the unrealistically severe loading condition imposed under test compared to that expected in practice, this should present no performance problems.

Application of Equation 1 to the average failure loads results in the following allowable racking loads/metre.

For JD5 framing:

$$R_D = 1.87 \text{ kN/m}$$

For JD6 framing:

$$R_D = 1.45 \text{ kN/m}$$

Possible practical allowable racking loads, taking account of the fact that edge distances to nails could well be the minimum value of 7 mm, are:

JD5 framing: $R_D = 1.75$ kN/m
JD6 framing: $R_D = 1.25$ kN/m

For the JD5 framing this results in an edl of (1.75 x 2.7) kN = 4.73 kN $>$ 3.75 kN the design load used in the test. However, the average racking deflection at this load is only about 3.2 mm.

For the JD6 framing an edl of (1.25 x 2.7) kN = 3.4 kN $<$ 3.75 kN the design load used in the test. The average racking deflection at this load is about 4 mm.

## REFERENCES

(1) Structural Plywood Wall Bracing - DESIGN MANUAL - Plywood Association of Australia, Ltd Publication.

(2) Recommendations for the Testing of Roofs and Walls to Resist High Wind Forces - Technical Report No. 5 - Cyclone Testing Station, The James Cook University of North Queensland, Townsville.

(3) AS1720 - Timber Engineering Code, 1975.

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