D O M E S T I C   G A R A G E

STRENGTH DETERMINATION BY

PROOF LOADING AND ANALYSIS

by

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A Thesis presented to the Faculty of
Engineering of the University of
Adelaide for one third fulfillment
of the requirements for the Degree
of Master of Engineering Science

Civil Engineering Department
University of Adelaide

June 1978

This thesis was accepted in partial fulfillment of the requirements for the award of the degree of Master of Engineering Science after Mr. L.M. Nash performed satisfactorily in an oral examination.

DECLARATION

To the best of the candidate's knowledge and belief, this thesis contains no material which has been accepted for the award of any other degree or diploma in any University, and contains no material previously published or written by another person, except when due reference is made in the text.

L. M. NASH

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SUMMARY

Experimental and analytical investigations are presented which show the strength, including cladding, of a Domestic Garage.

Tests were carried out on the bare steel frame, wall and roof panels and the assembled Garage. The tests showed the large stiffening effect of the cladding when the Garage was subjected to horizontal loads.

Agreement was obtained between the strengths as determined

(a) by analysis

(b) by testing

but only by using in (a) a yield stress considerably higher than the guaranteed minimum. This higher yield stress was justified for this particular garage by material testing as shown in Appendix A.

ACKNOWLEDGEMENTS

The author wishes to express his appreciation for the invaluable criticism and advice given by Mr. G. Sved, Research Associate (formerly Reader) in Civil Engineering, University of Adelaide, P. Wyten and Sons for the supply of materials for the shear panel and bare frame tests and supply and erection of the Garage, the following students of the South Australian Institute of Technology for assistance in taking test readings and wiring up the strain gauges,

Bare Frame Tests - W. J. Offler and P. P. Varga

Shear Panel Tests - J. Jacobs and C. Condo

Clad Frame Tests - T. K. Mace and D. R. Thomson

and finally to Mr. C. Plumb for fabrication and erection of the loading rigs. All the tests were carried out in the Civil Engineering Laboratories of the South Australian Institute of Technology.

1. INTRODUCTION

This thesis examines by load testing and analysis the total strength of a steel framed galvanized iron clad Standard Domestic Garage.

Buildings of this style are normally designed with the steel frames supporting the total load. This method of analysis bears little resemblance to the actual structural action of the building. As the steel framework must deflect under load so the sheeting must also deflect and so will also be loaded. The sheeting will in fact support most of the lateral load, as will be shown in this thesis.

Therefore under horizontal load, framework member stresses may be only a fraction of the stresses indicated by analysis while other parts of the structure, namely sheet-purlin fasteners and sheeting may be overstressed.

The sheeting offers large resistance to in-plane loads only and as this is a trussed building in which negligible horizontal deflections occur under vertical loading the sheeting contributes a negligible amount to the structural strength under vertical loading.

In order to obtain all the necessary information the load testing reported in this Thesis has been done in two parts:

(a) Components, framework and cladding

(b) Assembled Garage

Part (a) was to determine the stiffness of the basic components and their strengths in isolation while Part (b) was to determine the overall strength of the Garage.

This particular type of Garage has never been justified by analysis to the satisfaction of the Adelaide Metropolitan Councils and has only received building permission as a result of a proof-load test carried out by the S.A. Institute of Technology, Techsearch in 1974. The Techsearch load test was not instrumented and so no results other than the actual proof load are available i.e. no information is available on the actual distribution and magnitude of stresses. The Garage was not even tested to destruction as the client refused permission for this.

2. REVIEW OF LITERATURE

There has been a large amount of work done on frame-cladding interaction by various researchers but the main reference I used was the work done by E. R. Bryan (Reference No. 1) from Salford University, England. The research work published up to 1972 was very adequately summarised by S.J. Lawrence (Reference No. 4) in his Thesis on Stiffening Effect of Cladding on Light-Weight Structures.

Since 1972 Bryan's first volume of his work has been published and the second volume is almost ready for publication.

Bryan's book provides actual designs based on the flexibility method but was not totally relevant here because

(a) Sheeting profiles were different from those used in his work.

(b) Fasteners were not all trough fixed as roof sheeting was fixed through the crests.

(c) Sheeting side laps were not fastened.

(d) Purlin cleats were not stiffened.

The reasons above show that the garage as tested was not truly a stressed skin design as used by Bryan and other authors so therefore the literature on this topic serves only as a guide.

The same limitation applies to the other work carried out in Australia as this was primarily based on resistance to dynamic loads and in particular to cyclonic conditions.

Any load on the roof sheeting must be transferred to the ground and this is done by the roof spanning to the end walls which in turn transmit the load to the ground. So when the building has two end walls the roof sheeting acts as a beam transferring half the load to each of the end walls which in turn act as cantilevers and transfer the load to the ground (see Figure 2.1). If the building has only one end wall then both the roof and the end wall act as cantilevers with all the roof load going to the end wall. In addition to shear a moment is set up from the transfer of the roof load and this is taken out by a force couple on the side walls (see Figure 2.2). A building with only one end wall is not the preferred type of building in terms of use of the sheeting and is not covered in the literature but is the type of building that this thesis covers. As far as the sheeting is concerned a building with only one end wall has only approximately half the strength of a building with both end walls.

The building shown in Figure 2.1 would have several frames spaced along its length. The distribution of the load between the frames and the sheeting depends on their relative flexibilities. Progressing from the ends of the building towards the centre the deflection and so the flexibility of the sheeting increases but assuming all the steel frames are the same, the flexibility of the frames remain constant and so the load carried by the frames increases. Eventually the length to breadth ratio increases to the point such that the sheeting deflects so much at mid span that the centre frame itself takes all the load.

Bryan's recommended maximum length to breadth ratios are

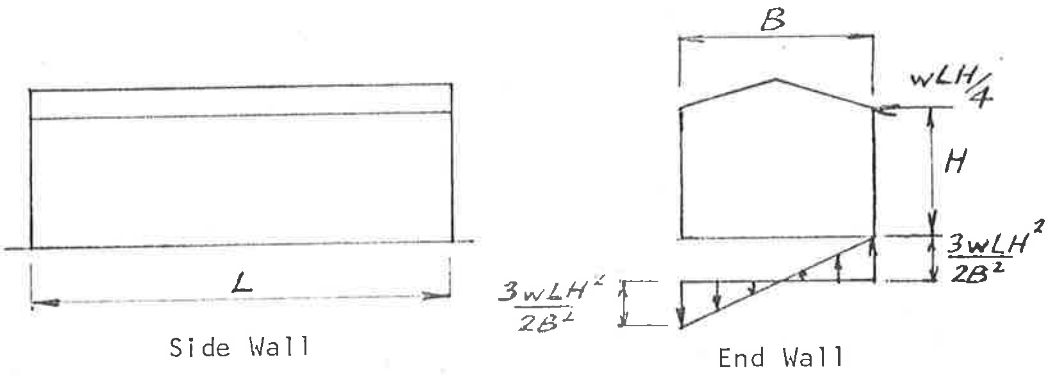
(a) Gable frame with both end walls 2.5:1

(b) Truss frame with both end walls 4.0:1

This would then suggest a possible maximum length to breadth ratio of 2:1 for a truss framed building with only one end wall. This is further reduced for a structure for which the sheeting has not been made as stiff as possible.

Stressed skin design does not necessarily reduce the cost of the building by the weight of the steel framework saved, as additional expense is incurred in stiffer cleats, lap fixings and trough fixings. The saving in the framework is largely in material as essentially the same amount of time is used for fabrication and erection.

A compromise solution appeals where stiffer cleats, lap fixings and trough fixings are not used and therefore no cost increase is incurred, but as the sheeting will still carry some load there will still be some saving in the steel framework and thereby some overall saving. This essentially was done in this thesis.



Note:

Both end walls fully sheeted so zero load on side walls.

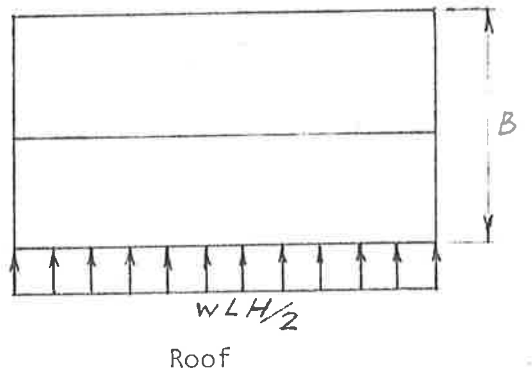
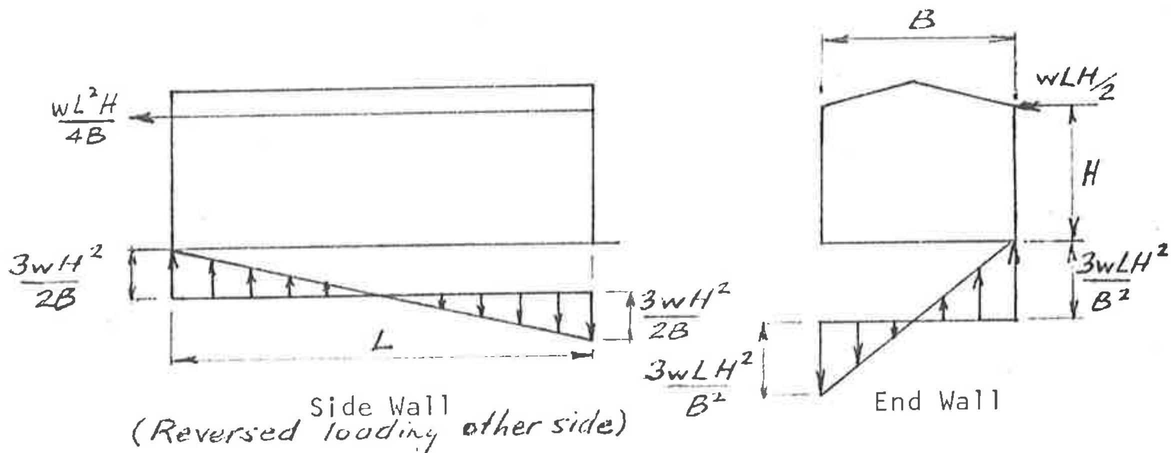


Figure 2.1



Note:

Only one end wall so side walls are loaded.

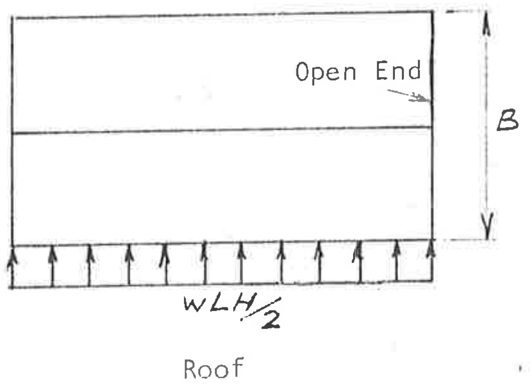


Figure 2.2

3. BARE FRAME TESTS

3.1 Introduction

This section deals with load tests on

a) Truss alone

b) Column alone

c) Steel Frame, truss plus columns.

These tests were necessary to determine stresses in members from known applied loads. The structure is indeterminate and needed several assumptions to be made before an analysis could be attempted.

These assumptions concerned

a) Degree of fixity of the truss to column connection

b) Column connection to truss

c) Degree of base fixity

d) Joint eccentricities

e) Battened column behaviour

f) Knee Brace behaviour

Therefore, idealising the structure by

a) Testing Truss only with true pin joint at one end and true roller joint at the other end

b) Testing Column alone with fixed base

c) Testing assembled frame

and then monitoring the strain on the anticipated critical members, enabled the determination of the actual stresses, bending moments and axial forces in the various members using simple bending theory. Then with the above uncertainties resolved an analysis was done and good agreement obtained between tests and analysis.

3.2 Preliminary Work

Prior to any testing considerable background investigation was carried out.

The frame as supplied by the manufacturer was accurately measured and details of the lack of straightness of members and eccentricities of connections noted as shown in Fig. 3.1.

A visit was made to the fabrication workshop to inspect the methods used. There it was noted that the lack of straightness of members in the truss was mainly due to the jig used for positioning truss members prior to welding.

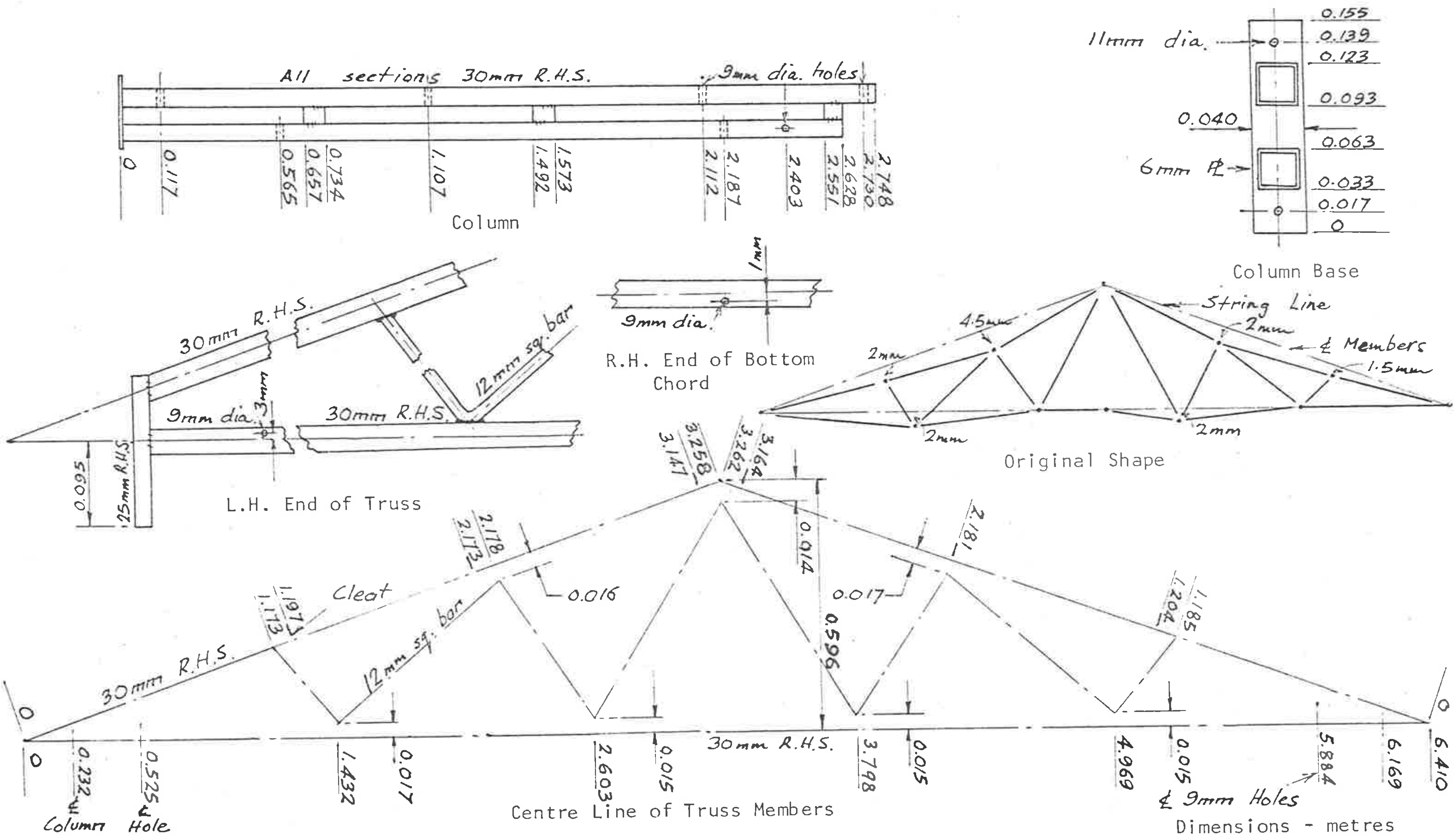
Another visit was made to Tubemakers of Australia Limited to obtain material specifications and to view testing procedures. Stress strain graphs from which yield stress and modulus of elasticity could be obtained were not available from Tubemakers. This was because it appeared that the specification varied over a wide range for different batches. Tubemakers recommended values of $F_y = 250$ MPa and $E = 2 \times 10^5$ MPa as being approximately correct for design purposes. Tensile tests were done on the frame materials supplied in order to obtain the design data. For details of these tests refer to Appendix A1.

3.3 Specifications

The frame consisted of steel truss, column and kneebraces. Complete frames were connected laterally by timber purlins and girts.

Member sizes as measured were:

main truss chords and columns	30mm x 1.56mm thick, square tube
truss ends	25mm x 1.56mm thick, square tube



All sections 30mm R.H.S.
9mm dia. holes
0
0.117
0.565
0.657
0.734
1.107
1.492
1.573
2.112
2.187
2.403
2.551
2.628
2.730
2.748
Column
11mm dia.
0.155
0.139
0.123
0.093
0.040
0.063
6mm PL
0.033
0.017
0
Column Base
30mm R.H.S.
12mm sq. bar
9mm dia.
3mm
30mm R.H.S.
0.095
25mm R.H.S.
L.H. End of Truss
1mm
9mm dia.
R.H. End of Bottom Chord
String Line
4.5mm
2mm
2mm
℄ Members
1.5mm
2mm
2mm
Original Shape
3.147
3.258
3.262
3.164
2.178
2.172
0.016
0.014
2.181
0.017
1.197
1.173
Cleat
0.596
1.204
1.185
12mm sq. bar
30mm R.H.S.
0
0
30mm R.H.S.
0
0.232
0.525
1.432
0.017
2.603
0.015
3.798
0.015
4.969
0.015
5.884
6.169
6.410
℄ Column
℄ Hole
℄ 9mm Holes
Centre Line of Truss Members
Dimensions - metres

Figure 3.1

knee-braces	16mm x 1.6mm thick, square tube
internal truss webbing	12.2mm square bar
timber purlins	60mm x 40mm
timber girts	40mm x 60mm

Size (mm)	Area (mm^2)	Moment of Inertia (mm^4)	Extreme Fibre Distance (mm)
30 square tube	177.5	24000	15.0
25 square tube	146.3	13450	12.5
16 square tube	92.0	3224	8.0
12.2 square bar	148.5	1850	6.1

The truss consisted of 30mm square tubing for top and bottom chords (with a $10^{o}34'$ angle of pitch), connected by 12.2mm square bar internal lacing. The internal members were welded to the outside chords. The top and bottom chords were connected at the extremities by 25mm square tubing placed vertically 236mm from the intersection of the centrelines of these members.

This 25mm tube dropped into a 30mm tube which separated the two 30mm tubes forming the columns. There was no means of locking or bolting this connection. The truss was held to the column by a 16mm tubular kneebrace at either end with 5mm bolts. The column had a 6mm thick base plate, which was held down by two 8mm 4.6 S bolts. Purlins and girts were positioned by 6mm coach bolts.

3.4 Load Tests

3.4.1 Truss

True pin and roller supports were fabricated for these tests. A single point load of 1.81 kN was placed at the second panel point and the truss was supported at the centre line of the columns and the centre line of the knee braces as shown in Figure 3.2 This test permitted the measurement of the truss stresses uncomplicated by the effect of the column-truss connection.

3.4.2 Column

The fixed base column was loaded by a single point load of firstly 0.20 kN and secondly 0.57 kN at an eccentricity of 1.0m as shown in Figure 3.3. This test gave the column stresses under a known moment plus a small axial load.

3.4.3 Assembled Frame

Six load cases were investigated using two loading systems, two types of bases with the frame, a braced and unbraced column, all as shown in Figure 3.4. All loadings were applied in the plane of the frame either sideways or normal to the roof cladding surface. From these elementary load conditions it was possible to evaluate numerically the combined effects for different types of wind loading.

Both pinned and fixed bases were used because it was considered that the true insitu condition lay somewhere between these two states and would vary according to soil properties and climatic conditions. It was thought advisable to work with a known fixity rather than to devise a system to attempt to simulate an actual possible condition.

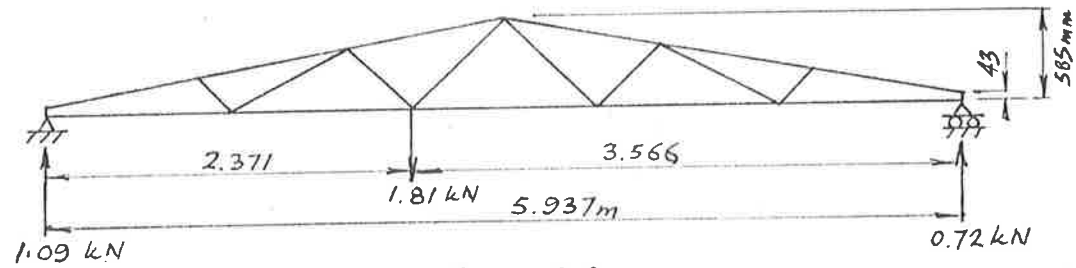


Figure 3.2

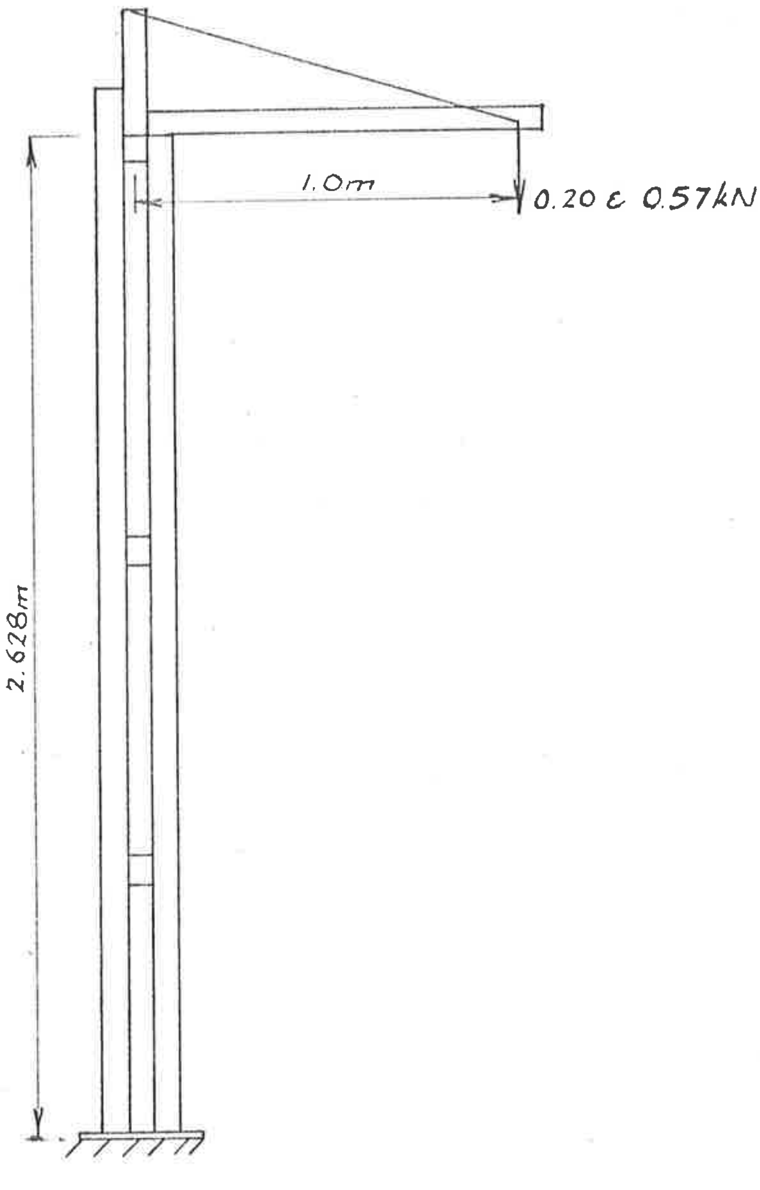
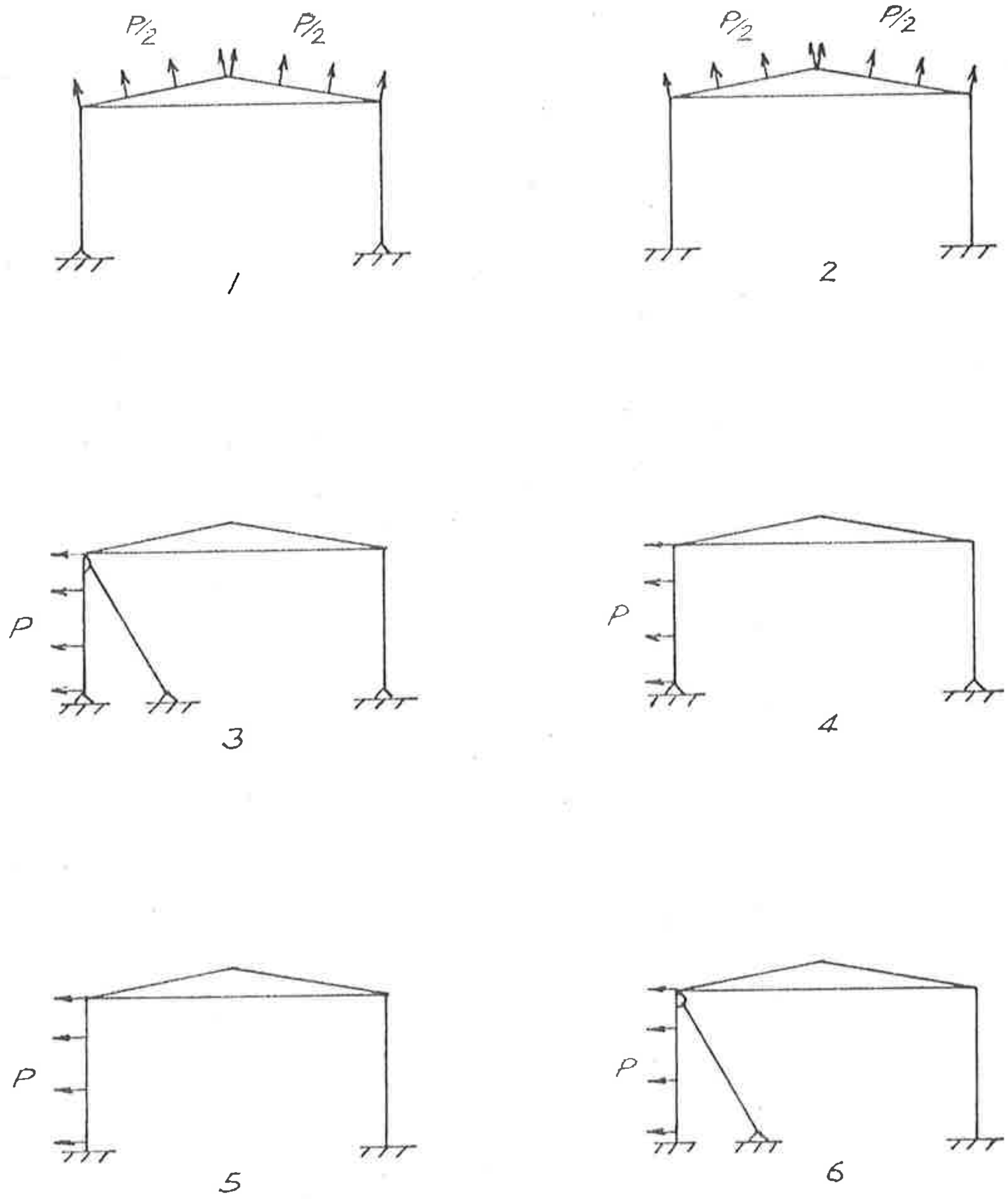


Figure 3.3



Note:

(1) Large Load Cell used for Cases 1 - 2
(2) Small " " " " " 3 - 6

Load Cases

Figure 3.4

Applied total loads in kN were

Case 1 P= 1.90, 3.79, 5.72, 7.65, 8.53, 6.86, 4.22

2 P= 7.96, 5.07, 2.60

3 P= 1.28, 2.03, 2.97, 3.90, 1.05

4 P= 1.01, 2.12, 1.43

5 P= 0.98, 1.86, 0.29

6 P= 1.34, 2.66, 3.28, 1.81

Main cases analysed were (1), (4), (5) of Fig. 3.4 as these gave the stiffness of the unclad frame. The other load cases were tested mainly for comparison purposes and to give an indication of the modified frame behaviour.

3.5 Bases

Two pairs of interchangeable bases of equal height were made:

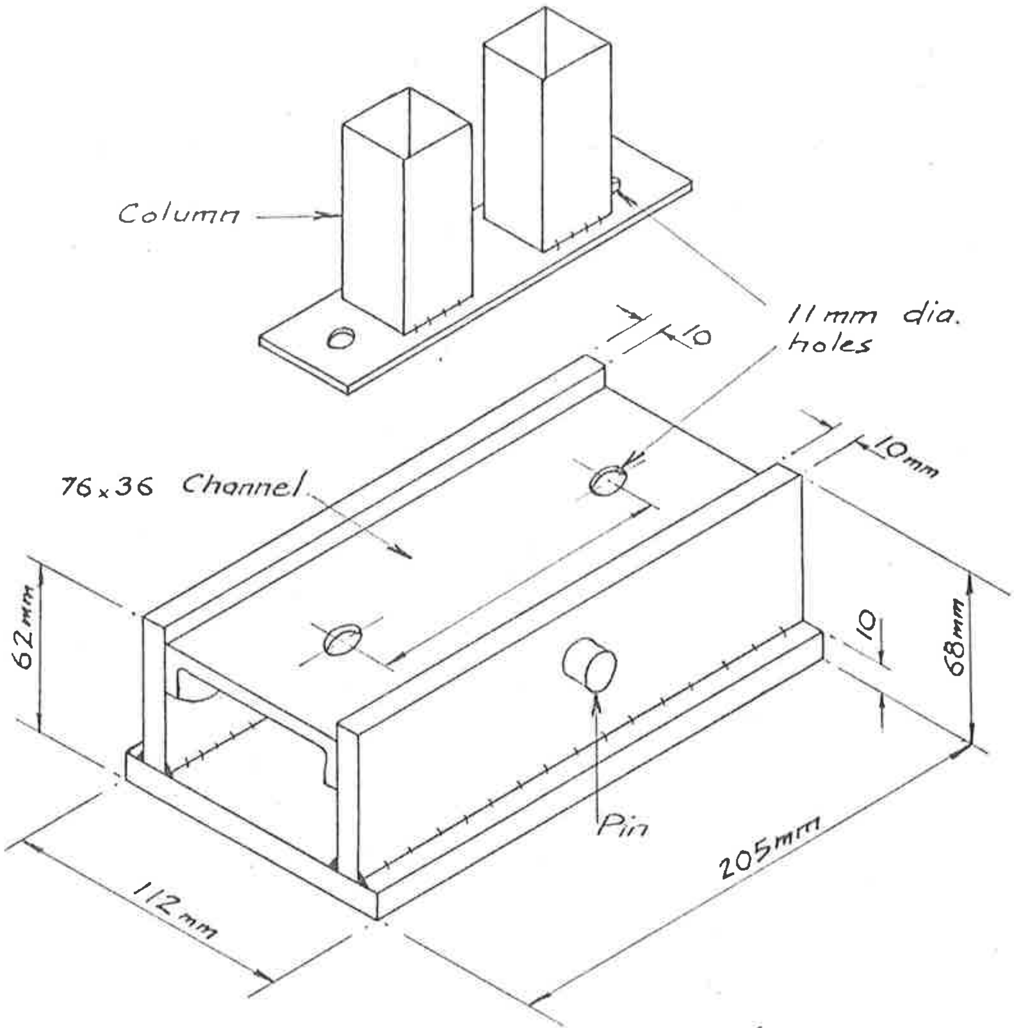
(a) fixed bases

(b) pinned bases free to rotate in the plane of the frame

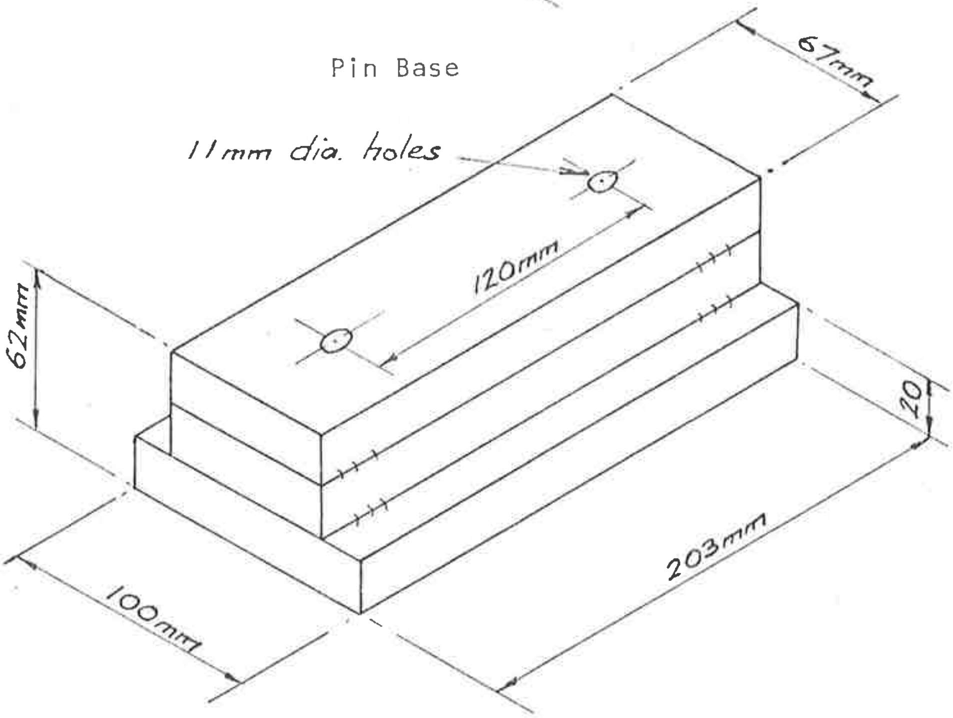
These bases were bolted to a 410 U.B. which in turn was secured to the Strong Floor in the Civil Engineering Laboratory with three 30mm diameter bolts in the proximity of the bases and at midspan. For details of Base-plates see Fig. 3.5.

3.6 Assembly

These series of tests investigated forces acting in the plane of the frame only, but since frames were normally connected by timber purlins and girts allowance was made for this effect. To simulate the lateral restraint provided by these members the standard connections were used on the frame and it was restrained in the direction normal to the loads by connecting the purlins and girts to the laboratory wall at a distance



Pin Base



Fixed Base

Isometric View of Frame Bases

Figure 3.5

of two metres. The wall connections were pinned to a hundred millimetre diameter water pipe of similar configuration to the garage frame. The wall support frame was secured to the laboratory wall columns. The pinned connections allowed the rotation in the plane of the frame and a small amount of rotation perpendicular to the frame. For layout of frame see Fig. 3.6 and wall connections see Fig. 3.7.

3.7 Loading

3.7.1 Equipment list

Enerpac hydraulic pump

Plessey Aust. Pty. Ltd. jack

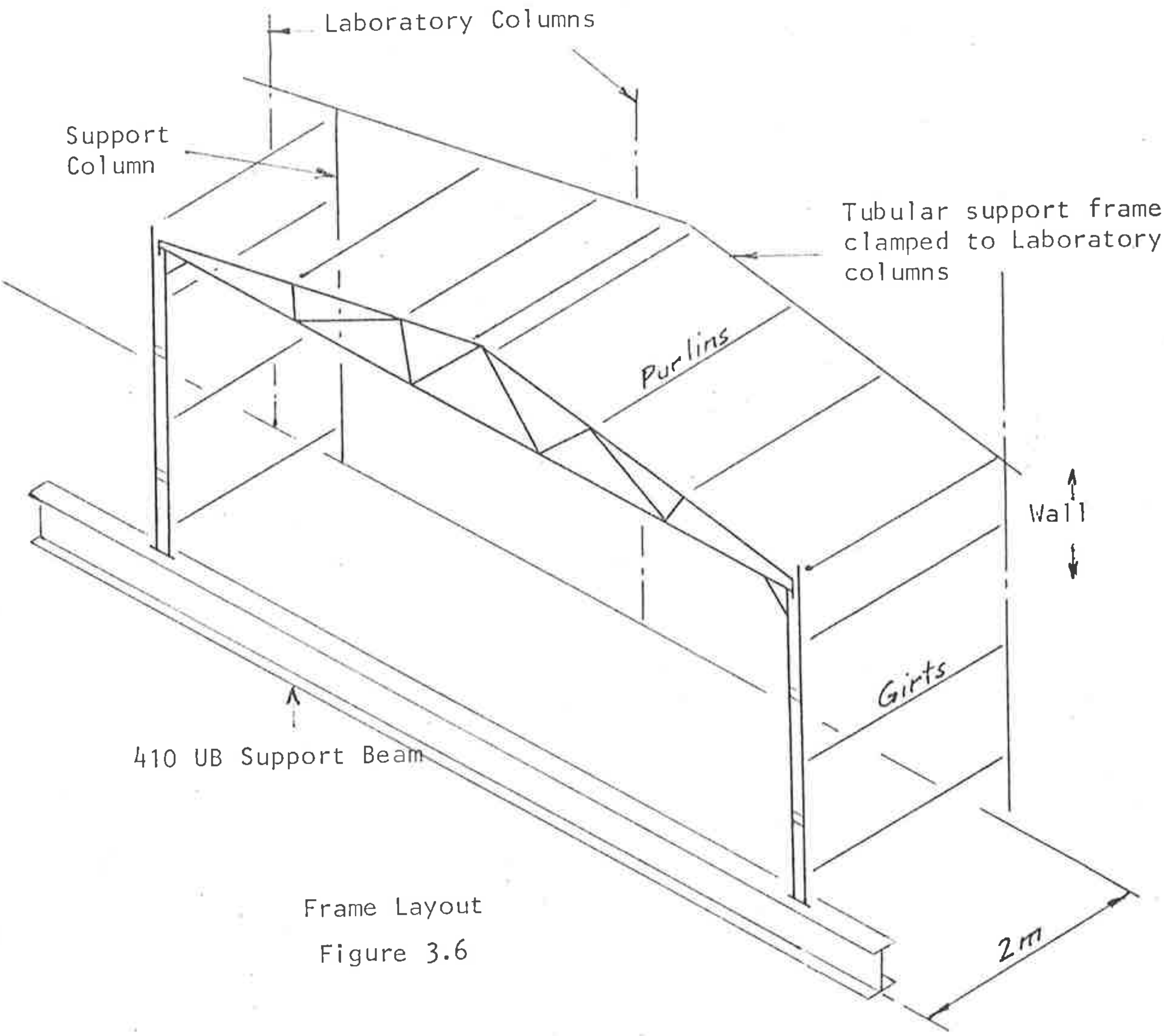
10mm wire rope

12mm diam. steel rod

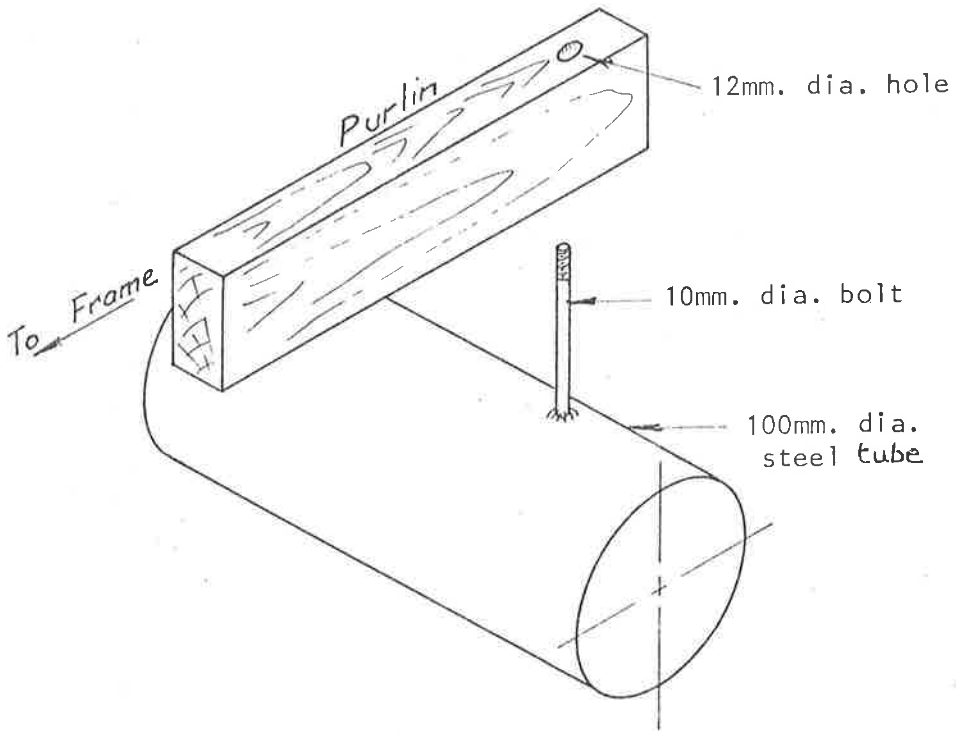
3.7.2 Load Mechanism

The uniform load which normally acts on the sheeting was transferred to the purlins and girts, resulting in point loads on the frame. The value of these concentrated loads were directly proportional to the area of sheeting supported by the respective member. That resulted in each panel point load being of a different magnitude. To simulate this a loading mechanism, which by means of different lever-arm lengths; distributed the total applied loads in the correct proportions was used. With this type of mechanism it was possible to apply the total load using a single hydraulic jack.

Loads were measured by load cells specially made and calibrated for these tests.



Frame Layout
Figure 3.6



Purlin Tube Support
Figure 3.7

For the side load cases the dead weight of the loading mechanism was supported by wires connected to an independent frame anchored to the Strong Floor. For this mechanism see Fig. 3.8.

For the upward loadings, the force was measured directly at the point where the total load was applied, thus avoiding any mechanical losses which might have occurred while transferring the force from the loading mechanism to the hydraulic jack. The weight of the loading mechanism was compensated for by zeroing the load cell whilst it was carrying the loading mechanism's self weight. See Figure 3.9.

3.8 Instrumentation

3.8.1 Equipment List

Schlumberger Solatron Data-Logger System

(a) Solatron 3317 Gauge Power Supply

(b) Solatron Analogue scanner

(c) Solatron Compact 33 Data Logger

(d) Solatron A210 Digital Voltmeter

(e) Facit Computer Tape Recorder

(f) Facit Type-Printer

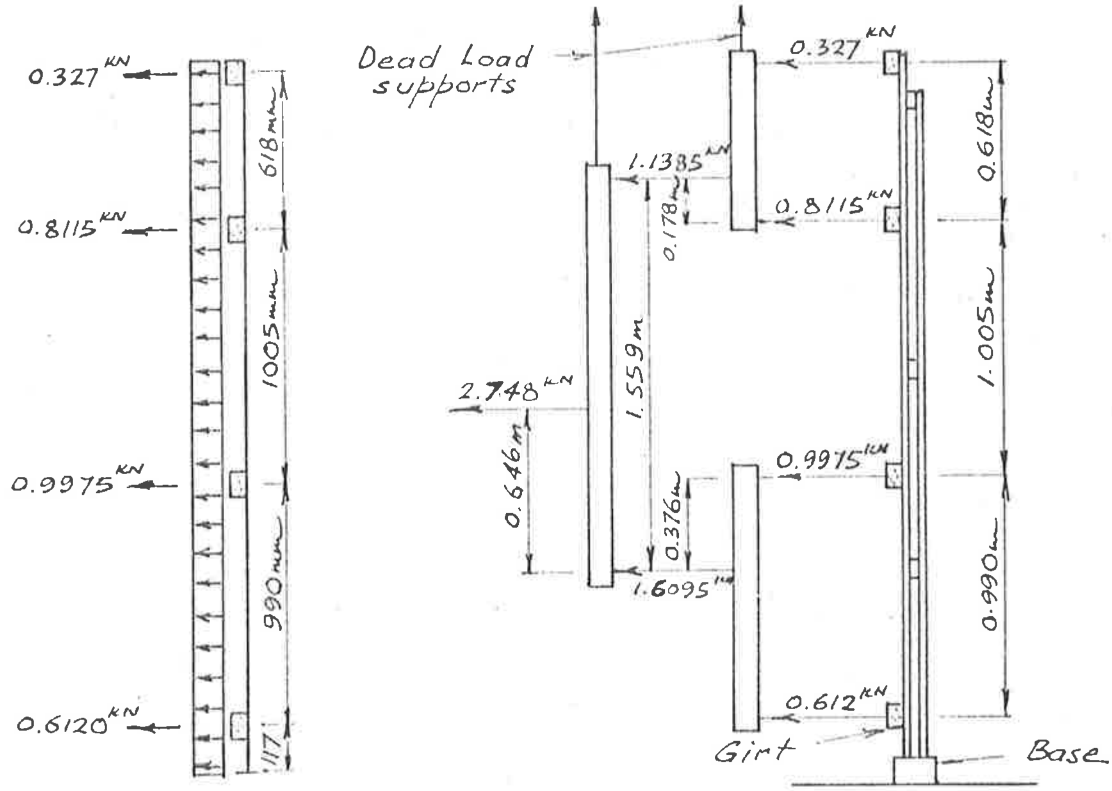
SystronDonner Digital Multimeter (Model 7205 Series no. 760)

Strain Gauges

3.8.2 Strain Gauges

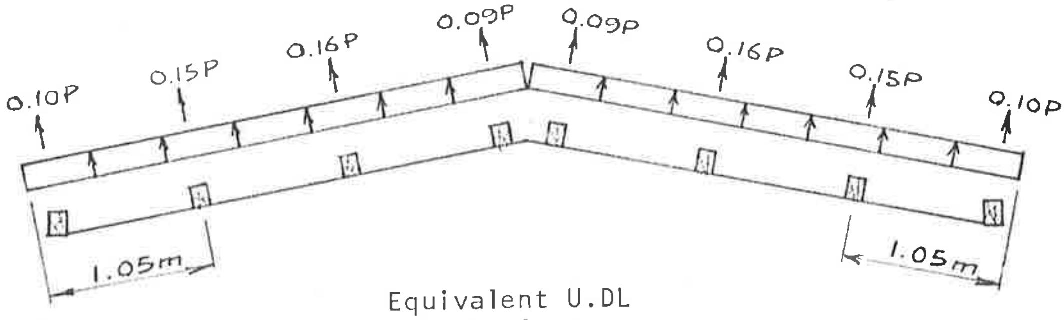
Kyowa KFC-5-C1-11, 120 ohm resistance strain gauges were stuck to the frame members on the two opposing faces, top and bottom, on the centreline longitudinal axis.

The strain gauges were concentrated about the truss-column joint, since this was the critical area. Other gauges were placed on the frame in positions which could be used to monitor the general frame behaviour. In total, fifty-four

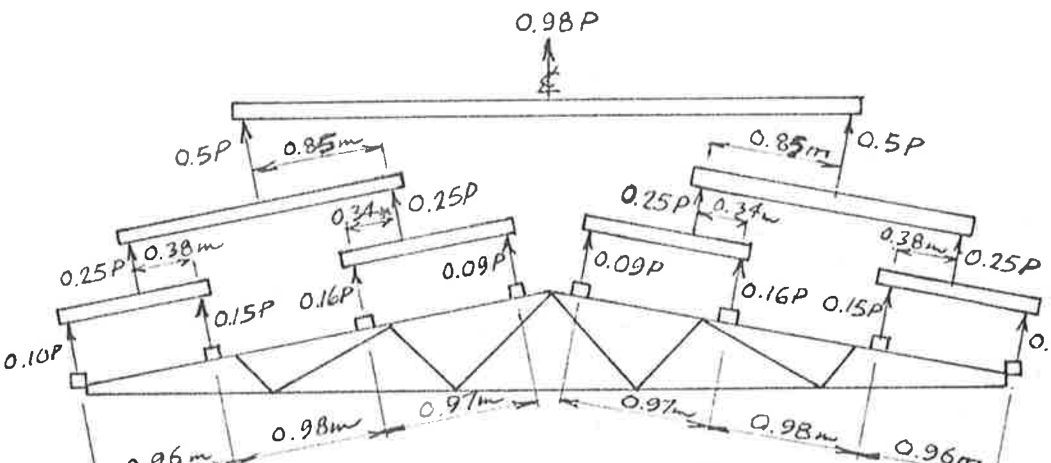


Side Load Details for 1 kN per Metre

Figure 3.8



Equivalent U.DL
on Purlins



Vertical Load Details

Figure 3.9

strain gauges were dispersed over the frame in such a manner that bending moments and axial forces could be found for the critical members.

In order to avoid local stress concentrations the strain gauges were placed approximately three member sizes from the node points. Position of gauges is shown in Fig. 3.10.

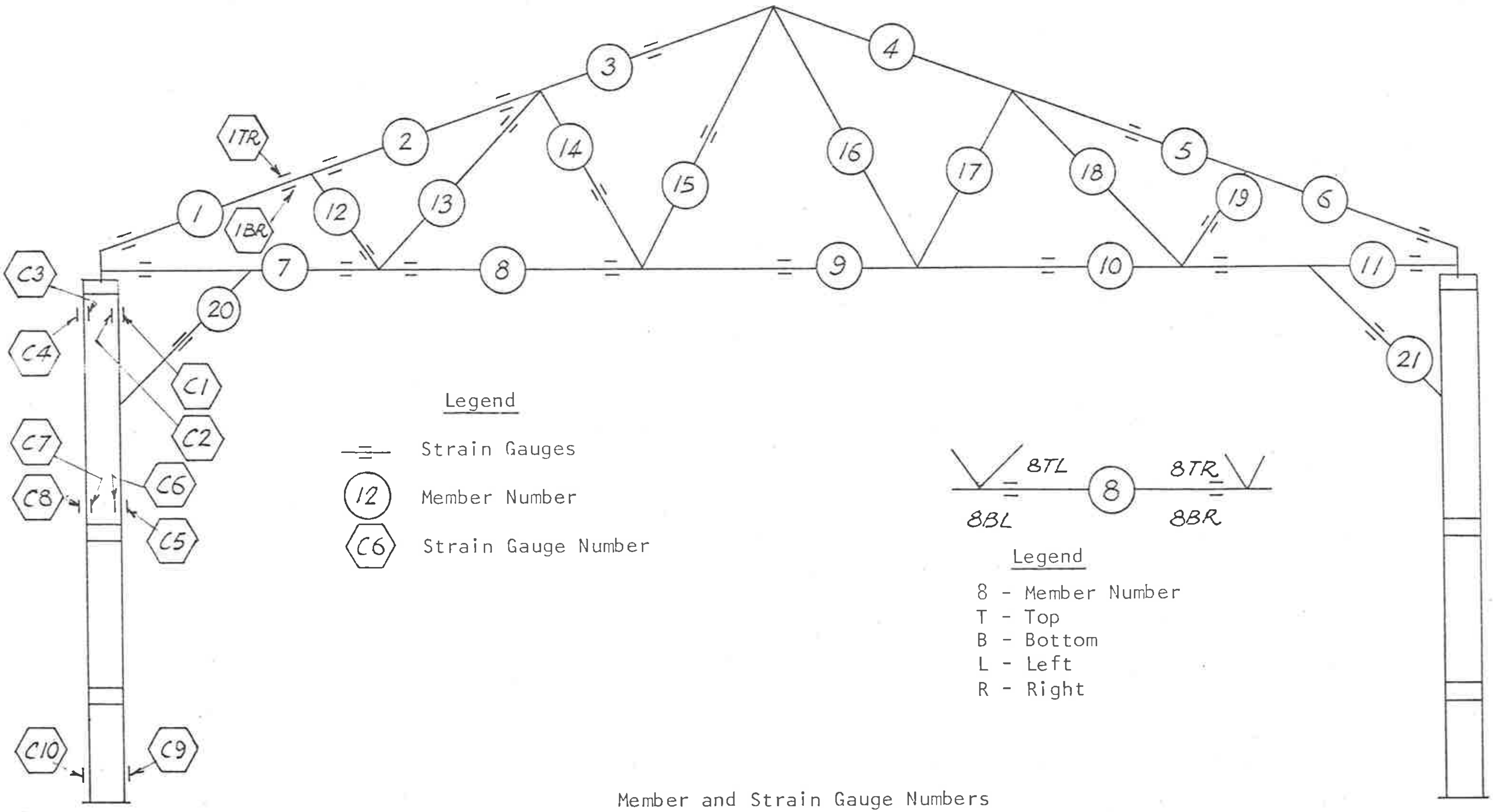
3.9 Recording Results

This was done electronically using the equipment listed in 3.8.1. This was the first use of this particular equipment and required considerable preparatory work before a start was possible. Also a large amount of electrical wiring was necessary to connect the strain gauges.

After extensive teething problems were overcome the system finally operated most satisfactorily.

The readings were recorded as printed values using the Facit Printer and as punched data on computer tape using the Facit Computer Tape Recorder. The great advantage of this system was that the results recorded on tape were then fed into the computer and used as data on the specially written computer program which gave as output the strains, stresses, axial forces and bending moments in the frame. As a result several load increments were used for every test.

Deflection readings were observed by dial gauges at the centre of the truss bottom chord and the top of the column.



Member and Strain Gauge Numbers

Figure 3.10

3.10 Results

The test results were too numerous to be included either here or in the Appendix but they are available for perusal. The key results were used in Appendix D - Structural Review and will be mentioned there.

3.11 Summary

As expected, it was observed under uplift loading that the truss lifted away from the columns until the slack in the kneebrace joints was removed. At this stage the truss was not supported at the columns but at the kneebrace connection which not being at a panel point, caused a bending moment in the truss generally but particularly in the bottom chord. In fact it caused the highest moment in the entire frame. Therefore the truss would be strengthened if bolted direct to the column.

The frame was very flexible under side load and particularly the pinned base frame. With pinned bases, the column deflected about 90 mm under a load of 1 kN.

Although the fixed base deflected much less, 76 mm for 1.9 kN, the base plate weld cracked under the load of 1.9 kN. This supported the opinion that the frame could not be considered to have fully rigid bases.

These tests showed that unless the sheeting could support most of the lateral load the bare frame would be unsatisfactory and entirely inadequate for resisting the structural loads.

4. SHEAR PANEL TESTS

4.1 Introduction

This section deals with load tests on a roof and wall panel plus all individual components of these panels. These tests were necessary to determine ultimate strengths and flexibilities of these parts. These test results could then be compared with the tests on the clad and unclad frames in order to determine the load sharing between the cladding and the steel framework.

Before attempting to ascertain the amount of deflection a complete wall or roof panel will undergo when subjected to shear loadings, loading behaviour patterns of individual components must be known. The series of tests described in this section were carried out with the following points in mind. The overall shear flexibility of a penel will be an accumulation of:

(i) Cladding deformation

- (a) distortion of corrugated profiles
- (b) elastic strain of sheeting

(ii) Cladding to framework fastenings

- (a) purlin cleats
- (b) nail fasteners in timber framework
- (c) local crushing and tearing of sheeting around nail fasteners

This enabled a complete solution of a panel's flexibility to be obtained rather than just an overall result.

4.2 Mechanical Properties of Cladding Material

4.2.1 Roof Cladding

The Roof Cladding comprised of standard Lysaght Custom Orb corrugated sheeting having a total coated thickness of 0.49mm.

4.2.1.1. Longitudinal Tensile Test

The test specimen consisted of a piece of sheeting approximately two corrugations wide x 300mm long. For details of test specimen see Figure 4.1.

The area of the specimen was found to be 83.0mm^2. An extensometer was used to measure the strain in conjunction with Demec points fixed to the sheeting with expoxy adhesive. A set of Demec points was fixed to both sides of the sheeting.

The results showed that the expected linear relationship existed between stress and strain.

The ultimate tensile load = 28.5 kN

Ultimate stress = 343 MPa

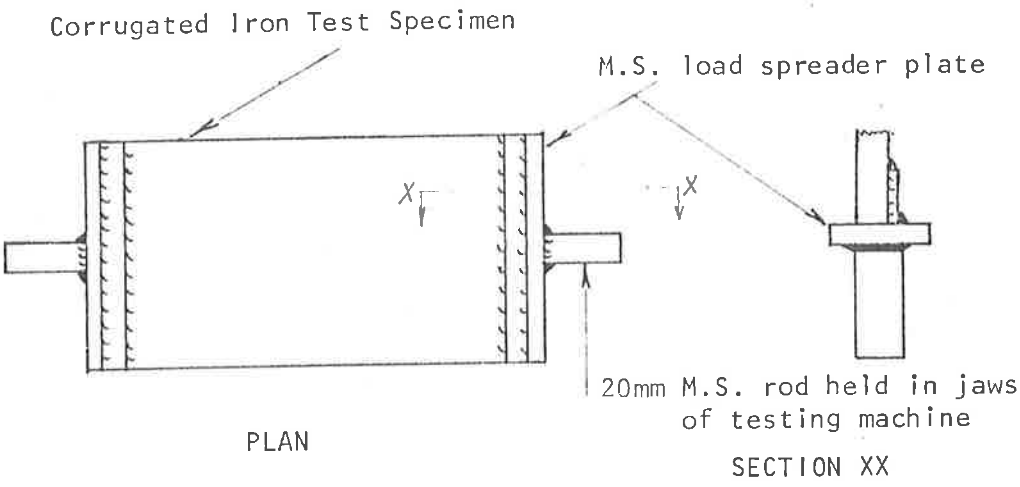
4.2.1.2. Traverse Tensile Test

This test was carried out to try and determine the strength characteristics exhibited by corrugated Custom Orb sheeting when subjected to a tensile load applied across the corrugations.

To enable this test to be carried out, a full sheet width was used for specimen length but the width of the specimen was limited to 42mm.

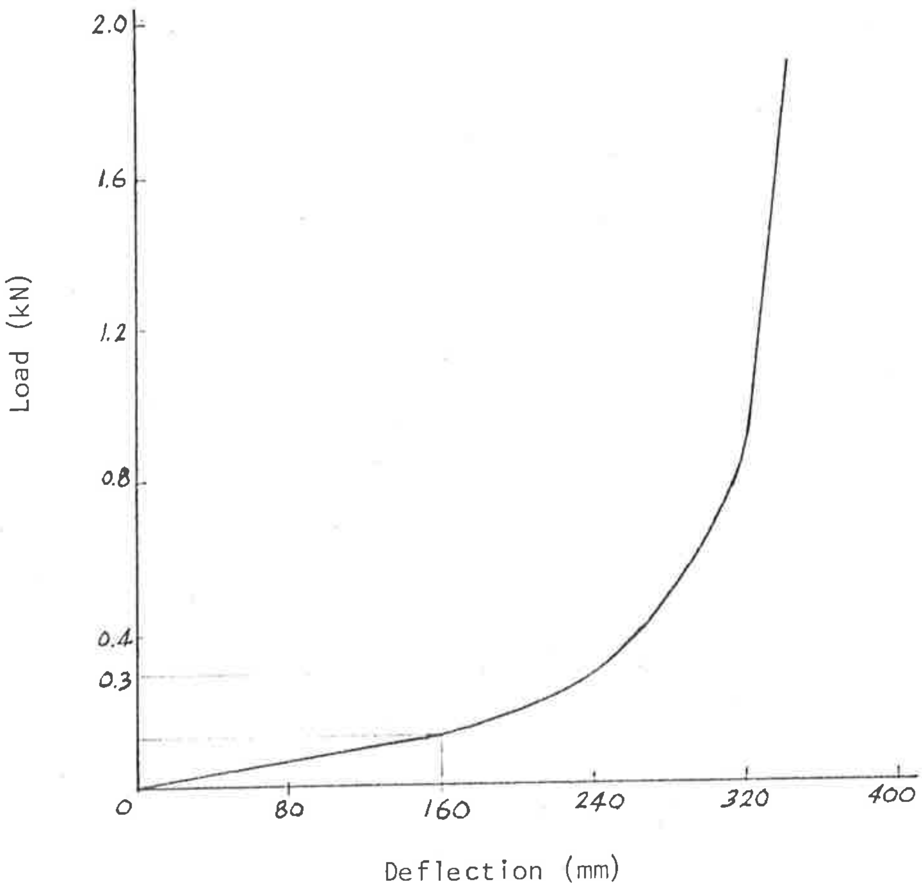
The results of this test can be seen on Figure 4.2.

From this particular test it can be seen that in the initial stages of loading an almost linear relationship existed between load and extension with considerable extension taking place up to about 0.3 kN after which, when the corrugations straightened out, a Hookean behaviour pattern was again observed.



Longitudinal Tensile Test Specimen

Figure 4.1



Transverse Tensile Test

Figure 4.2

This test showed that the sheeting offered little resistance to transverse loading.

4.2.2 Wall Cladding

The Wall Cladding used was of V Crimp type profile with the galvanised iron strip supplied by J. Lysaght Ltd and custom rolled by Cowells Steel Ltd.

The total coated thickness was 0.49mm.

4.2.2.1 Tensile Test

A profile section showing the size of specimen used in this test is shown in Figure 4.3.

The method of holding and transferring the tensile load across width of test specimen was the same as that used for the Custom Orb tests.

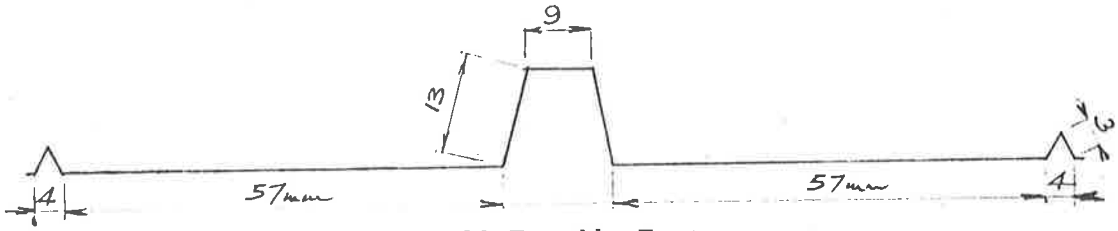
Cross sectional area of specimen = $78.24mm^2$. The average thickness of sheeting found by a micrometer was 0.48mm. Three sets (pairs) of Demec points were used to determine the strain by the use of an extensometer.

Location of these points is shown in Figure 4.4.

The ultimate tensile force was recorded as 22.7 kN giving an ultimate tensile stress of 290 MPa. From the resulting linear stress versus strain relationship the value of Young's Modulus was found to be 1.95×10^5 MPa.

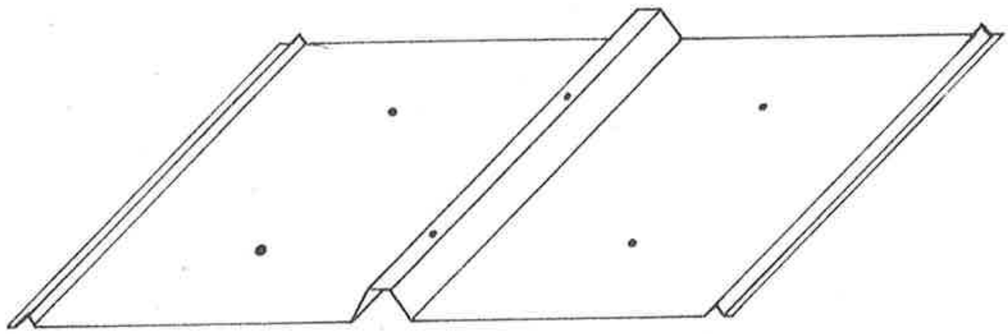
4.2.2.2 Transverse Tensile Test

This test was carried out in the same manner and for the same reasons as for the roof sheeting. Again test indicated little resistance to lateral loading, see Figure 4.5.



Wall Tensile Test

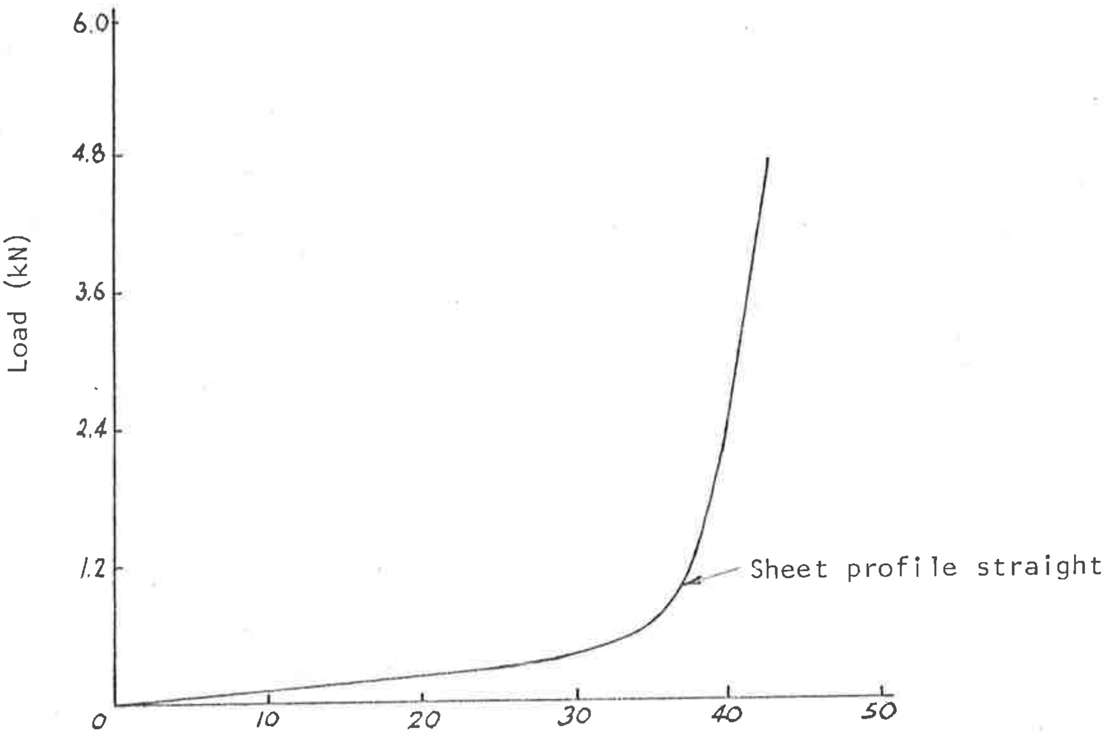
Figure 4.3



Demec Points

Wall Tensile Test

Figure 4.4



Transverse Tensile Test

Figure 4.5

4.3 Flexibility of Purlin Cleats

For arrangement see Figure 4.6.

From the outset it was obvious that with the timber purlin fixed in position the flexibility of the cleat connection would be considerably less when loaded as in Case A than when loaded as in Case B, due to a wedging effect between the timber purlin and the top chord of the roof truss.

Consequently two series of tests were carried out to determine the different flexibilities.

To simulate the actual loading conditions on the purlin/cleat connection with the roof sheeting, the loading cradle was suspended the distance equivalent to the depth of a sheeting corrugation (16mm) from the face of the timber purlin.

4.3.1 Purlin/Cleat Case A Loading (see Figure 4.7)

During this test deflections were measured at the end of the cleat and head of the roofing nail.

The results showed an almost linear relationship existed for both deflection recordings.

The resulting flexibilities were:-

(i) End of cleat = 0.95 mm/kN

(ii) End of nail head = 1.45 mm/kN

4.3.2 Purlin/Cleat Case B Loading (see Figure 4.8)

Again deflections were measured at the end of the cleat and nail head.

A flow type failure of the cleat was evident when a load of 440 N was applied. The resulting flexibilities are:-

(i) End of cleat = 1.75 mm/kN

(ii) End of nail head = 2.60 mm/kN

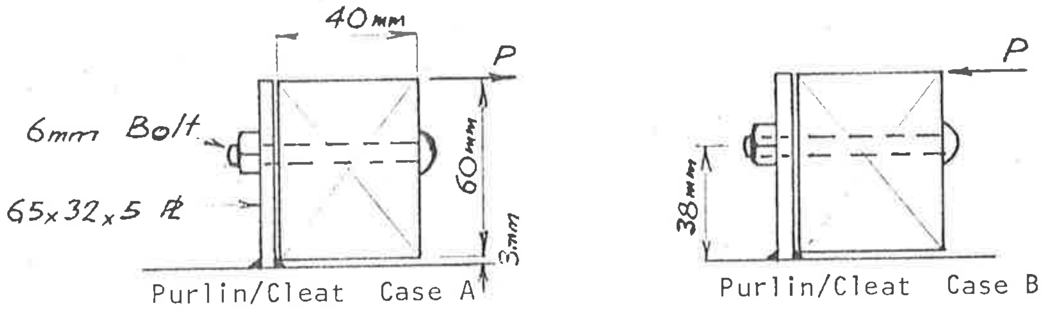
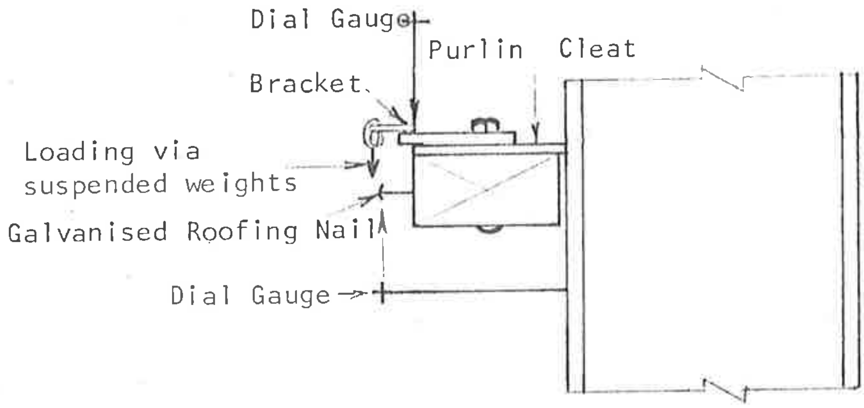
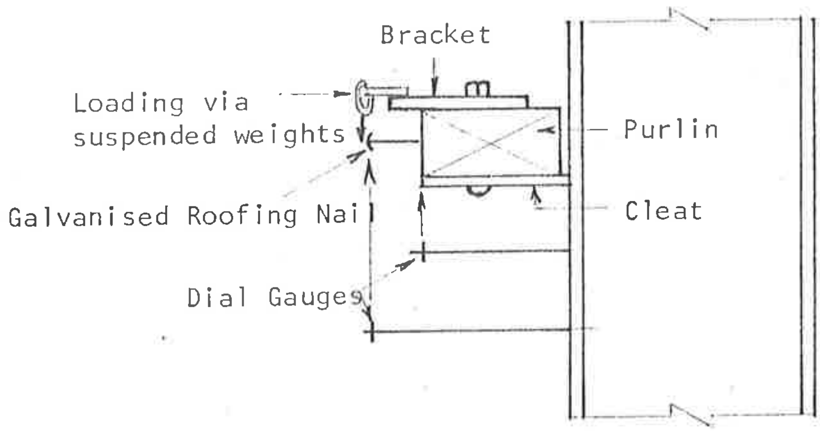


Figure 4.6



Purlin/Cleat Case A Loading

Figure 4.7



Purlin/Cleat Case B Loading

Figure 4.8

On comparing the corresponding flexibilities obtained from loading Cases A and B; the flexibility of the Purlin/Cleat connection for loading Case B was nearly twice that for Case A.

4.4 Testing of Nail Connection

4.4.1 Single Roof Cladding/Purlin Fixing

The fixing of the corrugated "Custom Orb" roof sheeting to the timber purlins was examined by using standard galvanised roofing nails, (see Figure 4.9).

Dial gauges were positioned to measure the deflection of the sheeting as well as the deflection at the nail head. It was hoped that the difference between these two readings would give the amount of localized crushing of the sheeting at the nail hole.

However, due to the difficulties experienced in trying to obtain a satisfactory fixing location at the nail head for the dial gauge only the deflection of the sheeting was recorded.

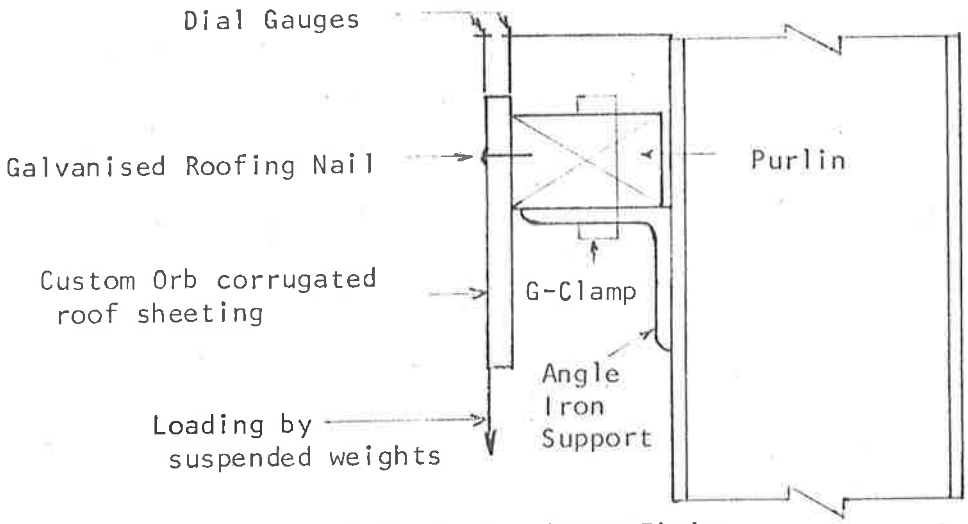
Results of this test can be seen on Figure 4.10.

Failure of this type of connection was by excessive bending of the nail.

The load capacity of the nail was observed to be 380 Newtons.

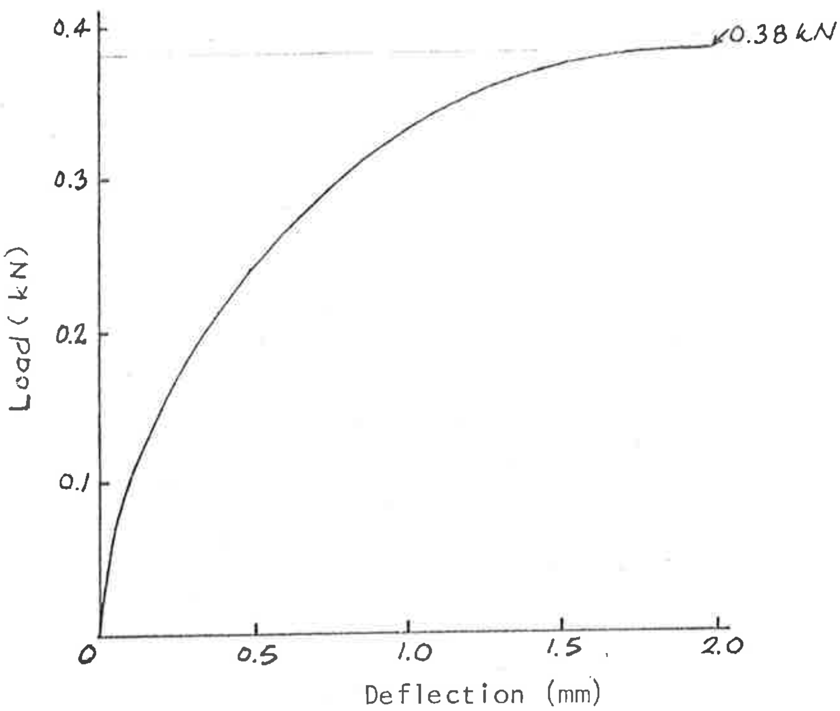
4.4.2 Shear Strength of Roof Sheeting/Nail Connection

The object of this test was to find out the shear load resistance of "Custom Orb" corrugated sheeting connected to the purlin by a single roofing nail. To avoid failure of the nail by bending the nail fixing was made in the trough of the corrugation with the load suspended from the end of



Nail Test - Crest Fixing

Figure 4.9



Nail, Crest Fixed

Figure 4.10

the sheeting. The test equipment used was identical to that used in the previous test. Excessive crushing of the sheeting was observed when the applied load was about 1.5 kN when slight creeping was also observed. Ultimate, rapid progressive tearing of the sheeting about the nail fixing occurred when the load reached 2 kN.

4.5 Roof Panel Test

For this series of tests the roof panel was placed on the rig upside down (i.e. purlins exposed on the top); this was done so that a load could be placed vertically to simulate the uplift load experienced by the roof.

The measurement of displacements was done by placing dial gauges in various positions as shown in Fig. 4.11.

The shear load was applied via a hydraulic jack and measured with a load cell. Application of normal load was achieved by placing 135 bricks on the roof panel.

Tests that were carried out on the roof panel were as follows:

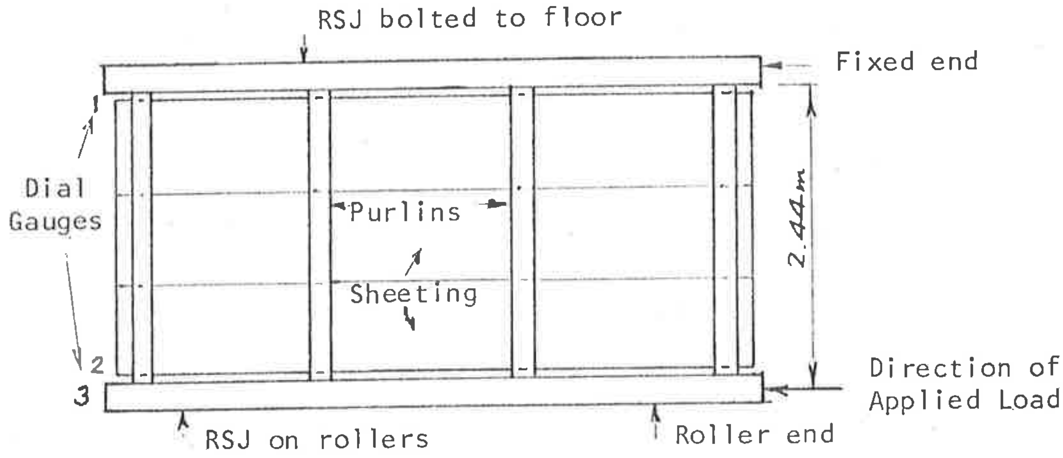
(1) Shear load plus normal load.

(2) Repetition shear load from one end without normal load. This test was carried out to establish the relationship between load and deflection (permanent deformation) after a load-release cycle.

(3) Loading from reverse direction.

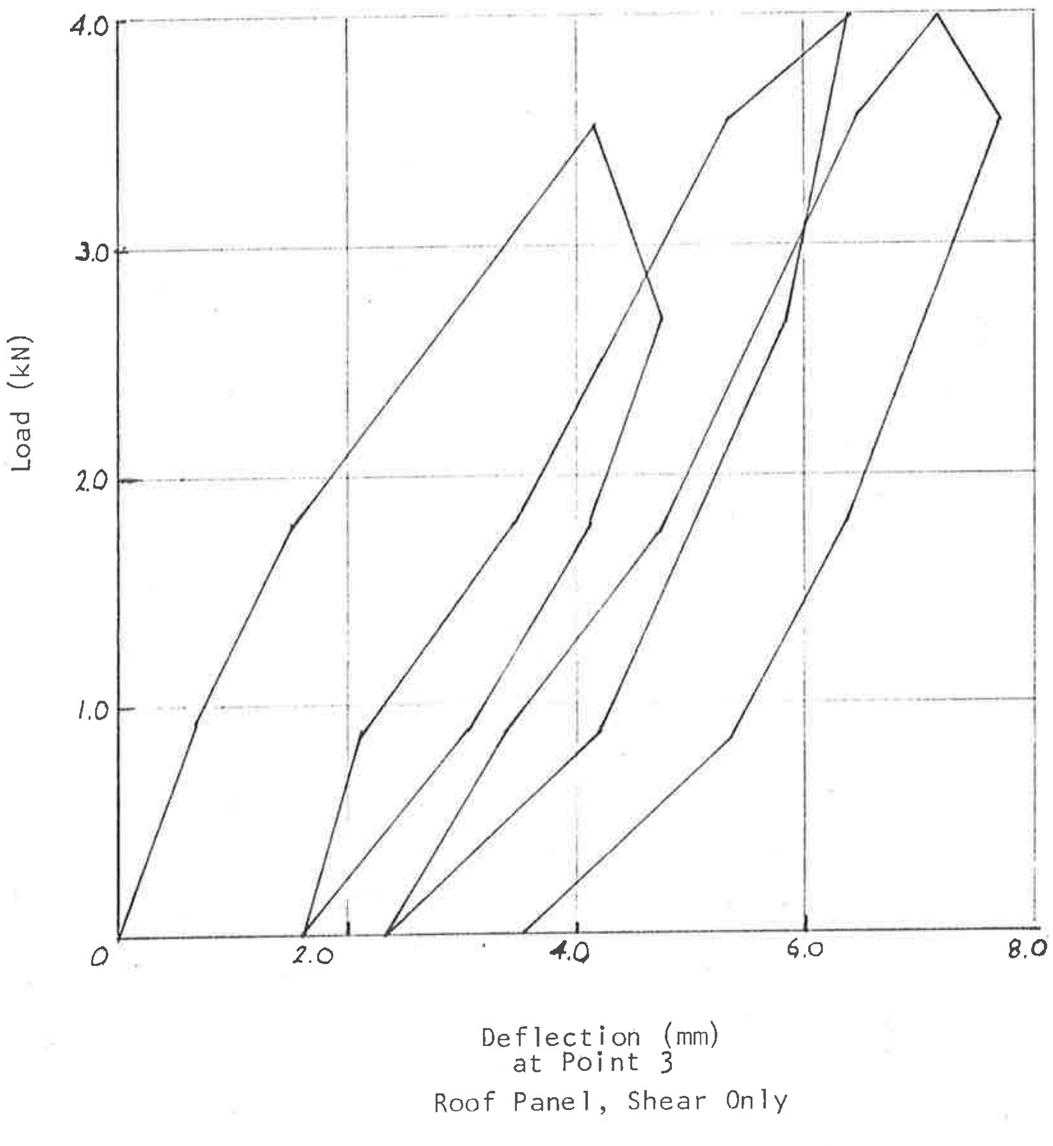
4.5.1 Repeated Shear Loads

Three cycles of loading and unloading were done and deflections noted. For load deflection graph at point 3 see Figure 4.12.



Roof Test Panel, 2.4m x 3.0m

Figure 4.11



Roof Panel, Shear Only

Figure 4.12

4.5.2 Shear Load plus Normal Load

A total of 135 bricks were placed on the roof to simulate a 5.18 kN load due to uplift.

One cycle of loading and unloading was done and deflections noted. For deflections at point 3 see Figure 4.13.

It can be seen that the addition of the vertical load only caused a small increase in both the maximum and the permanent deflections of the panel.

4.5.3 Reversal of Shear Load

For the next series of tests the direction of the shear load applied to the panel was reversed, this was done to simulate a change in wind direction so that the effects of

(1) Initial reversed loading

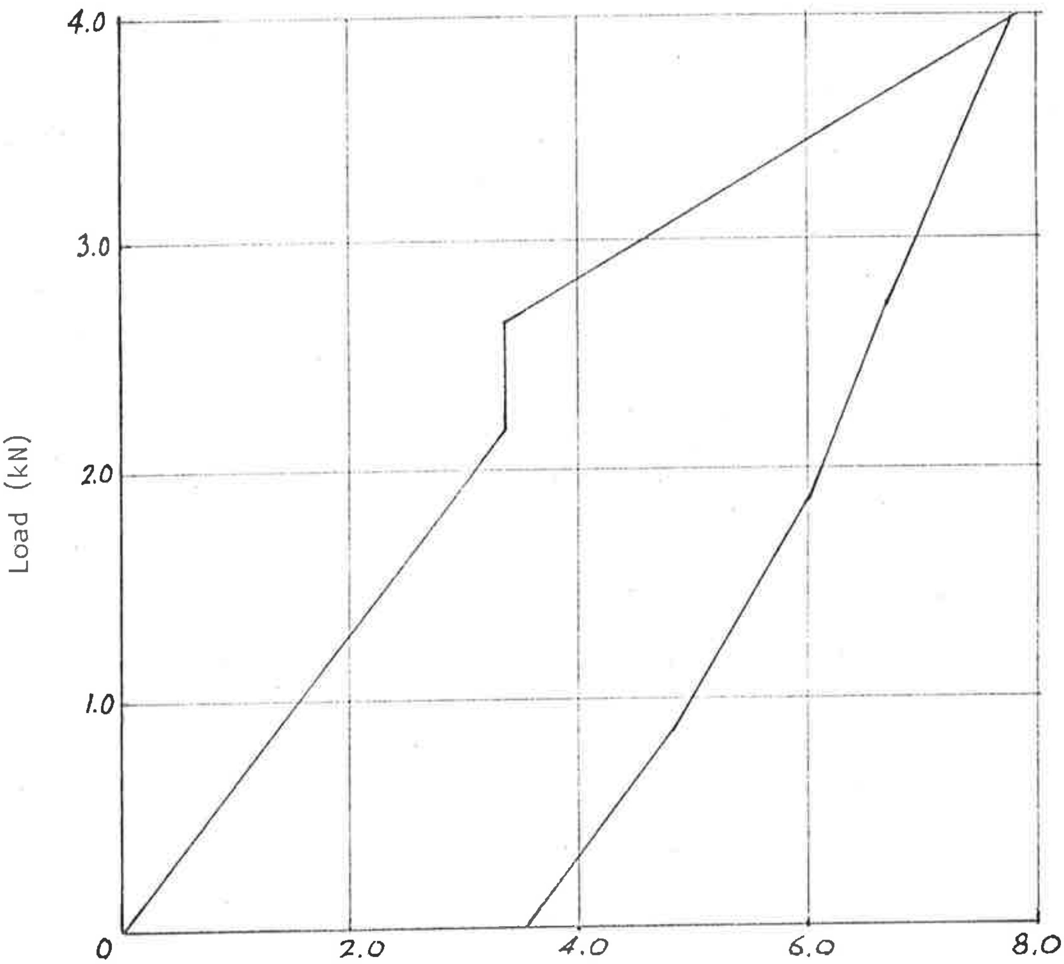
(2) Repetitive loading

could be observed.

Initial Cycle - curve for point 3 see Figure 4.14.

It can be seen that the deflection under the same load was far greater than before.

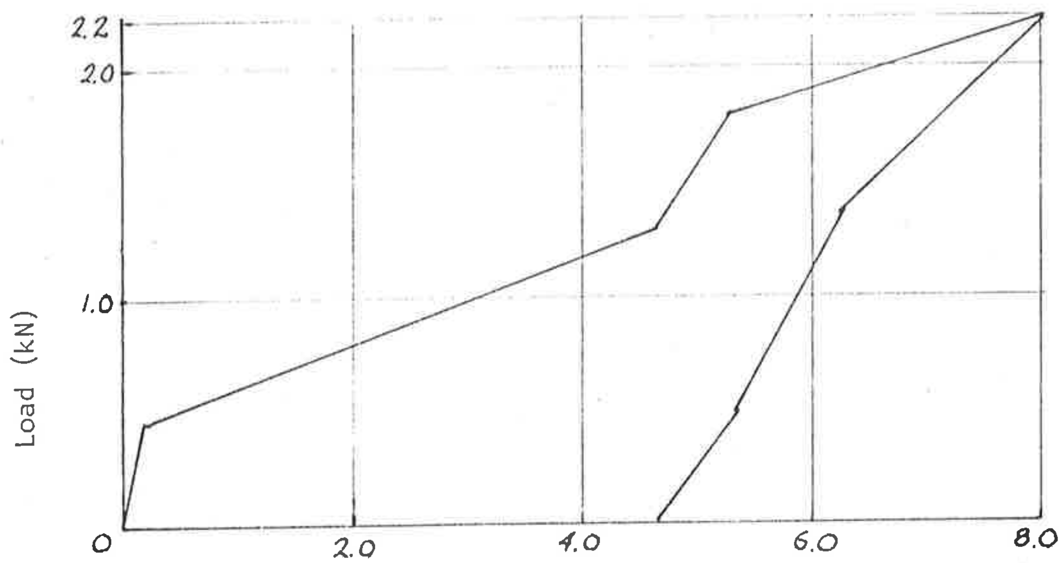
An explanation of this is that when the original load was applied the nails were bent over and they in turn tore the sheeting. The load was applied via the purlins and so when the load was reversed no contact between the sheeting and the nails occurred until the nails moved back across the tears in the sheeting. These larger deflections then resulted from this frame slackness.



Deflection (mm)
at Point 3

Roof Panel, Shear + Normal

Figure 4.13



Deflection (mm) at Point 3

Roof Panel, Shear Only but Reversed

Figure 4.14

Repetition Loading in the Reversed Direction

The curves here followed the same shape as for the original shear load, but with increased deflections due to slackness of the frame.

4.5.4 Discussion

On completion of the tests the sheeting was taken from the test rig and examined. It was noticed that tearing occurred all over but was particularly bad along the sheeting laps and was more severe towards the free support. The worst tearing occurred along the sheeting laps. This indicated that part of the load was transferred from sheet to sheet and where this transfer occurred tearing took place. It should also be noted that during testing the sheets moved relative to each other as shown in Figure 4.15.

The reason why tearing of the roof panel was significant was due to the fact that the nails were more severely loaded at the intersection of the two sheets. This caused larger deflections of these nails which in turn caused more severe tearing of the sheeting.

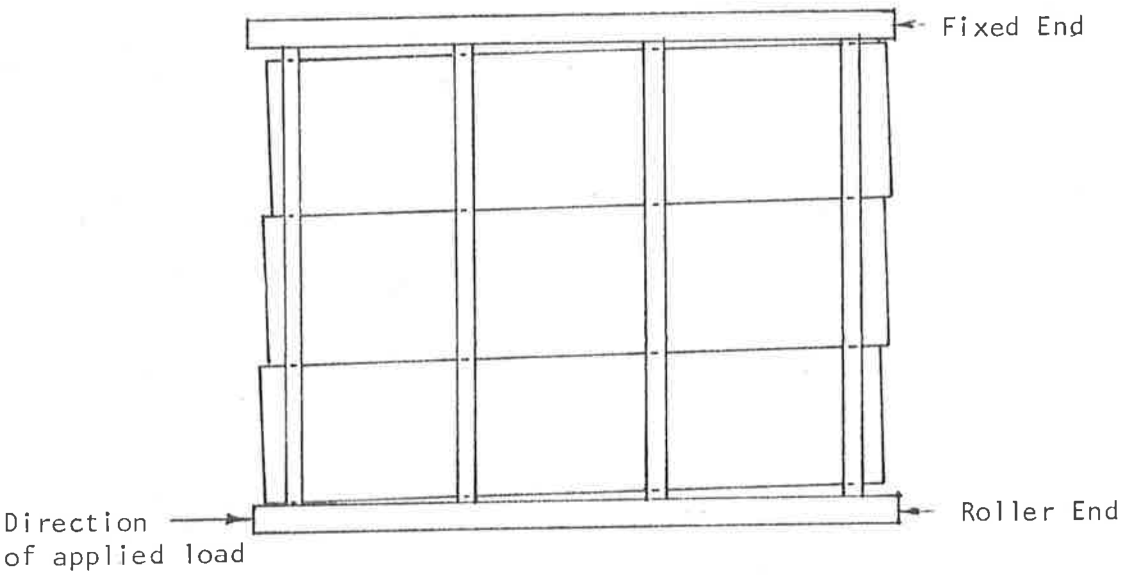
4.5.5 Ultimate Load

Ultimate capacity of this roof panel was 4 kN.

4.6 Wall Panel Tests

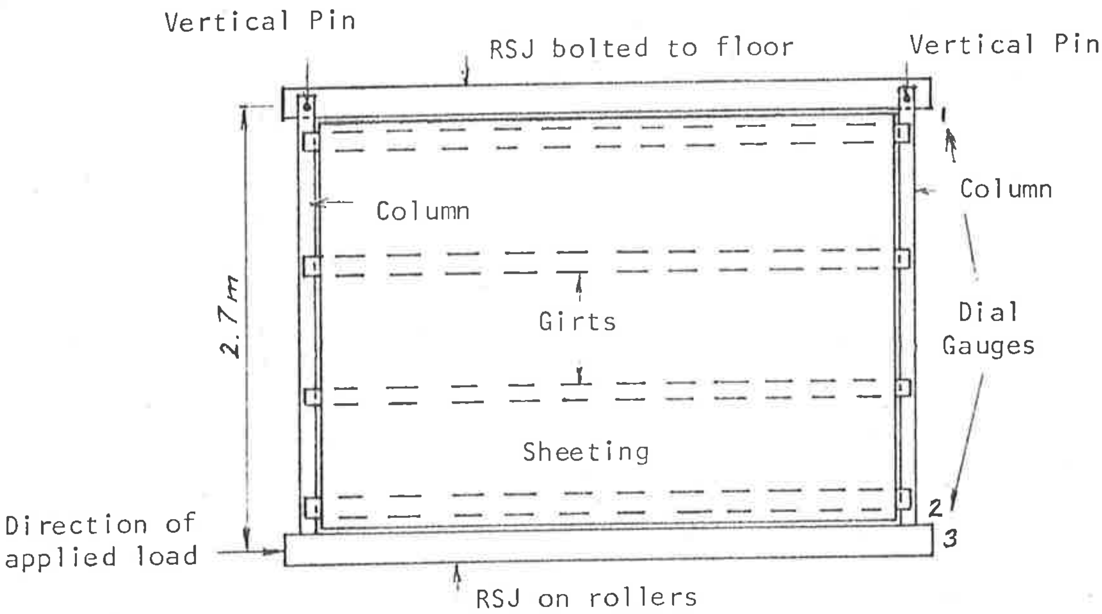
A different supporting rig was used here as shown in Figure 4.16.

Loading of the panel was as previously but without the normal load (i.e. suction).



Roof Test Panel, After Loading

Figure 4.15



Wall Test Panel, 2.7m x 2.44m

Figure 4.16

4.6.1 Repeated Shear Loads

As for 4.5.1. For end load deflection graph see Figure 4.17.

The results showed that the wall panel was stronger and stiffer than the roof panel.

4.6.2 Reversal of Shear Load

For this series of tests the load was applied from the reverse direction.

From this series of tests, two graphs were obtained:

(1) Initial deflection when load was first applied in the reverse direction. Figure 4.18.

(2) The repetition type loading on the panel from the reverse direction. Figure 4.19.

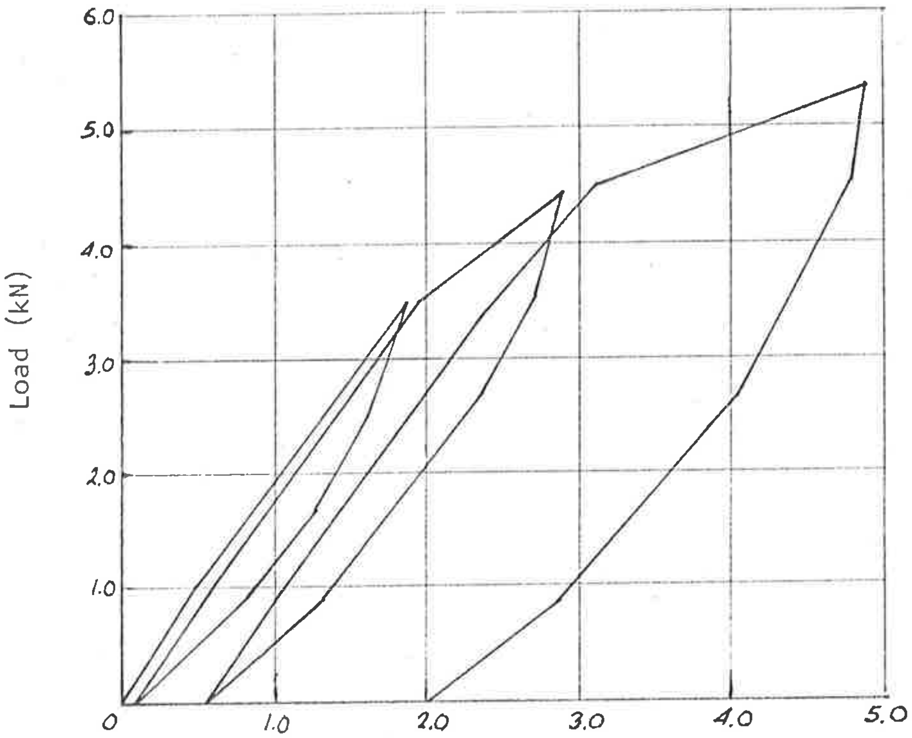
4.6.3 Ultimate Load of Wall Panel

Failure of the wall panel was achieved by applying a load in the reverse direction. Ultimate capacity of this wall panel was 5.33 kN.

During the test it was evident that the sheeting was being severely stressed because warping was taking place on the edge near the pinned edge.

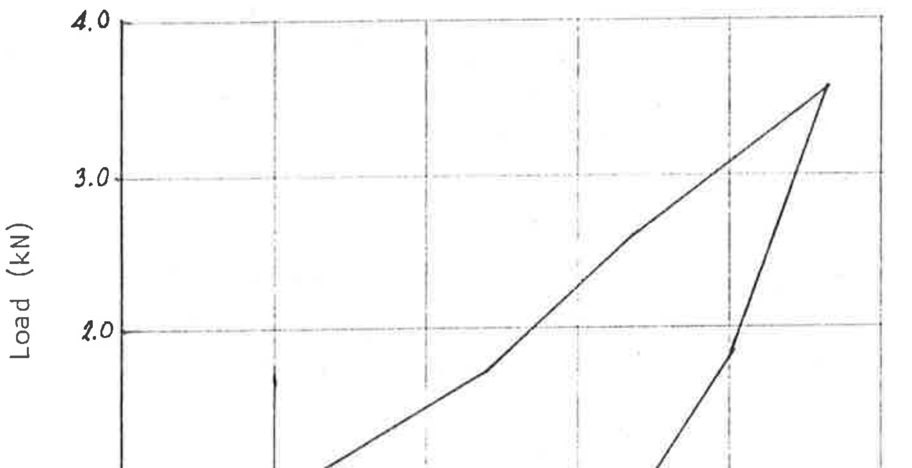
It should also be noted that each sheet displaced at right angles to the applied load as shown in Figure 4.20.

Other interesting observations were that the tearing of each sheet was opposite to that of the sheet before it. Here the nails were bent in the opposite directions, see Figure 4.20. Frame slackness also occurred here, similar to discussion of 4.5.3 but was less severe. This slackness was included in Fig. 4.18 but not Fig. 4.19 as the dial gauges were reset to zero for the last set of loads.



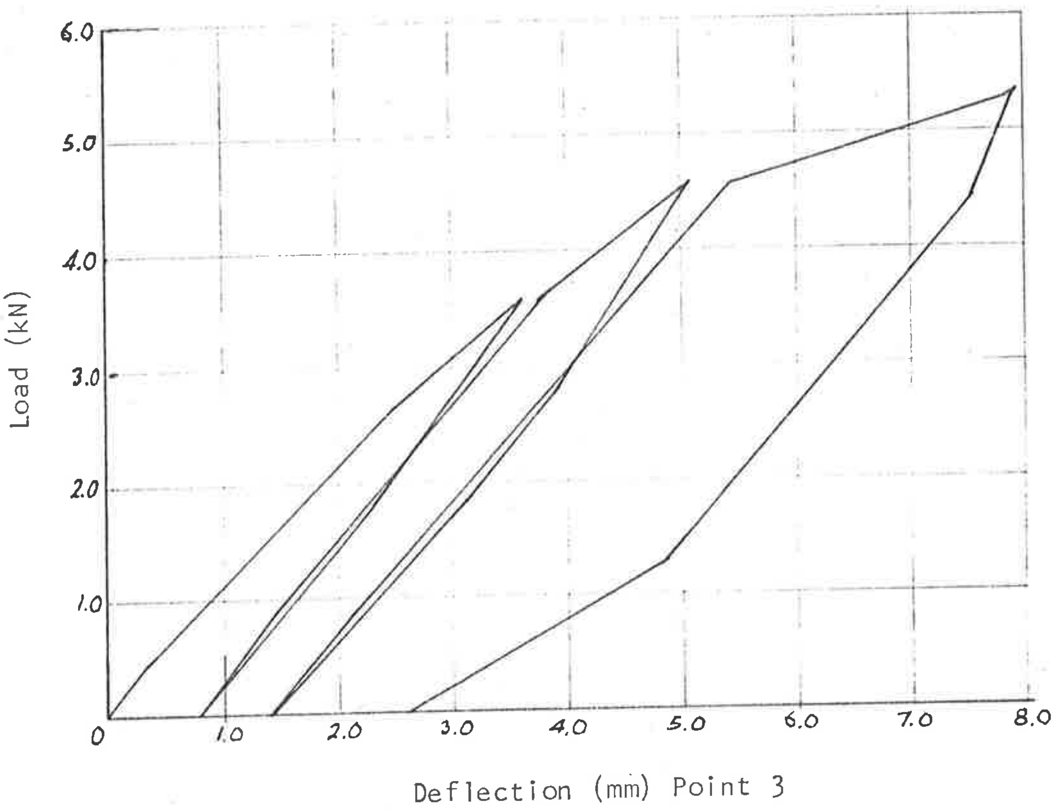
Deflection (mm)
at Point 3
Wall Panel

Figure 4.17



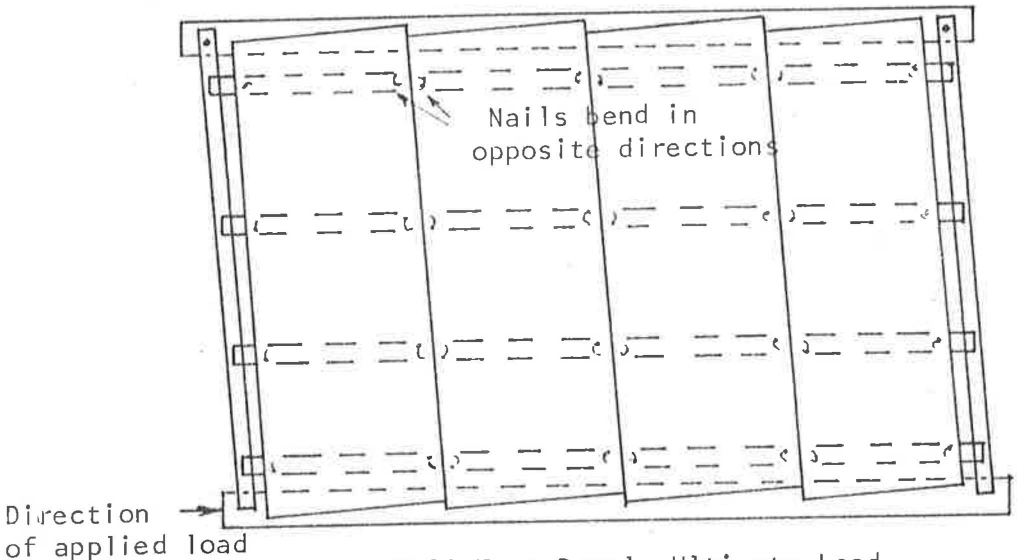
Deflection (mm) Point 3
Wall Panel, Load Reversed

Figure 4.18



Roof Panel, Reversed Loading

Figure 4.19



Wall Test Panel, Ultimate Load

Figure 4.20

4.7 Comparison of Results Obtained for Roof Panel with those from Wall Panel

As can be clearly seen in all the tests performed the total deformation for the wall panels was less than that for the roof, even though the load applied on the wall panel was higher than the load applied to the roof panel. This load difference was due to trough fixing the wall sheeting against crest fixing the roof sheeting which resulted in the roof nails acting as cantilevers. It was this cantilever action which caused severe bending in the roof nails and also the resulting eccentric loading on the roof sheeting that caused this greater permanent deformation.

4.8 Summary

The modes of failure exhibited by the wall and roof panel units during testing reinforced the recommendations made by E.R. Bryan as to how the flexibility of a shear panel can be decreased.

For both the wall and roof panel unit tests failure was evident by excessive slippage at side laps between sheeting, either caused by bending of the nail fixing or tearing of the sheeting at the nail fixings or a combination of both. It is obvious then that the number of nail fixings down the side lap were not sufficient. Bryan suggests that this deficiency may be eliminated by increasing the number of side lap fixings. Without decreasing the spacing of purlins and girts one method by which this can be done would be to use a 'pop' rivet type fixing.

On looking at the overall results from tests carried out on both types of panels, the flexibility of the wall panel consisting of V crimp sheeting was significantly less than the roof panel consisting of corrugated iron sheeting. Since both panels were of approximately the same dimensions and construction, this implies that the relative large flexibility value of the roof panel is caused by the bending failure of the nails. Again this reinforces the recommendation put forward by Bryan to the effect that nail fixings should preferably be made in the valley rather than through the crest where cladding having a rolled profile is used. Another precautionary measure suggested by Bryan is to stiffen the cleats against bending failure by using stiffening gussets. The tests carried out on the purlin cleats support this idea.

The sheeting can therefore be greatly stiffened by adopting Bryan's suggestions. This of course increases the cost as more fixings and stiffer cleats are required. Also trough fixing of roof sheeting produces sealing problems necessitating more expense.

Here a compromise has been used in which less than optimum use of the sheeting was set against increased cost of obtaining the maximum stiffness of the sheeting.

The final structure adopted for the steel frame had virtual pin joints at both ends of both columns i.e. a four-hinged frame. The steel frame then had no theoretical lateral resistance so the sheeting must take all the lateral load. The resulting deflection was tolerable and so an acceptable and economical design was achieved.

5. CLAD FRAME TESTS

5.1 Introduction

This section deals with load tests on the assembled garage. These tests were necessary to determine the ultimate strength of the structure and also to observe the member stresses from known applied loads. The test results on this clad frame could then be compared with the tests on the unclad frame and the wall and roof panels in order to determine the sharing of the load between the cladding and the steel framework.

The test loads were determined from Australian Standard Codes 1170, Parts 1 and 2. The garage frame has been shown by Section 3 to be incapable of carrying the lateral loads without significant assistance from the cladding so rendering the stressed-skin action essential for its survival. By monitoring the strains on the expected critical members, the relevant strains, stresses, axial forces and bending moments could be calculated using simple bending theory. This method of analysis had to be modified once the stresses reached the yield stress as simple bending theory no longer applied.

5.2 Specifications

Cladding

Roof, Lysaght Custom Orb	0.48mm sheeting
Walls, Lysaght V-crimp	0.48mm sheeting

Member Sizes, Section Properties and Frame Assembly

As per Section 3.3.

Overall Size and Details

There were three bays each of 2.44m making the overall centre line dimensions of span 5.94m, height 2.65m and length 7.32m. All four steel frames were of the same size and construction. One end wall is normally covered by sliding doors but for these tests the doors were removed leaving this end wall completely open except for the gable sheeting. The only other openings were a window in the other end wall and a personal access door in one side wall. All sheeting was nailed to the purlins and girts with spring-head nails all in accordance with Lysaght's recommendations.

All this information was recorded on drawings nos. DG1-8 drawn by D. J. Adams, checked by the author and dated October, 1977. These drawings are presented in Appendix E.

5.3 Load Cases

Initially wind loading was simulated by loading separately one side wall loaded inwards and outwards; the other side wall, both halves of the roof and the back wall loaded outwards only. This was done for both fixed and pinned bases. These results were examined and as expected from the Bare Frame tests, the knee braces were actually disadvantageous due to the severe bending stresses they caused in both the truss and column through not being connected at panel points.

Also comparing fixed and pinned base results, only marginal differences were observed pointing to a weakness in the base plate indicating that the fixed bases were virtually pin bases.

Therefore there were good reasons for discarding the knee braces, bolting the trusses direct to the columns and using pinned bases throughout.

Next Dead Load plus Live Load was tested for both knee braces in and out and negligible differences in stresses resulted. The knee brace had considerable slack in their joints and this therefore prevented them from acting under downward load.

It was therefore decided to do the final series of tests without the knee braces and with mainly pinned bases. The longitudinal wall bracing was also discarded.

Accordingly the twelve tests shown in Figure 5.1 were done.

The fixed base, if it offers any real help, will do so for side wind load and so this case was included. Also end wind load which was the critical case was done with a fixed base as the very small help this base offers can easily be satisfied in practice.

Testing for component loads as well as for combined loads was necessary to determine whether end wind or side wind was critical as the frame could only be loaded to failure once.

5.4 Bases

An adjustable base plate was devised by which a pinned base or a fixed base condition could be simulated (see Figure 5.2).

In the Bare Frame tests two separate bases were used. This was not satisfactory here so this modified base was used to produce both cases.

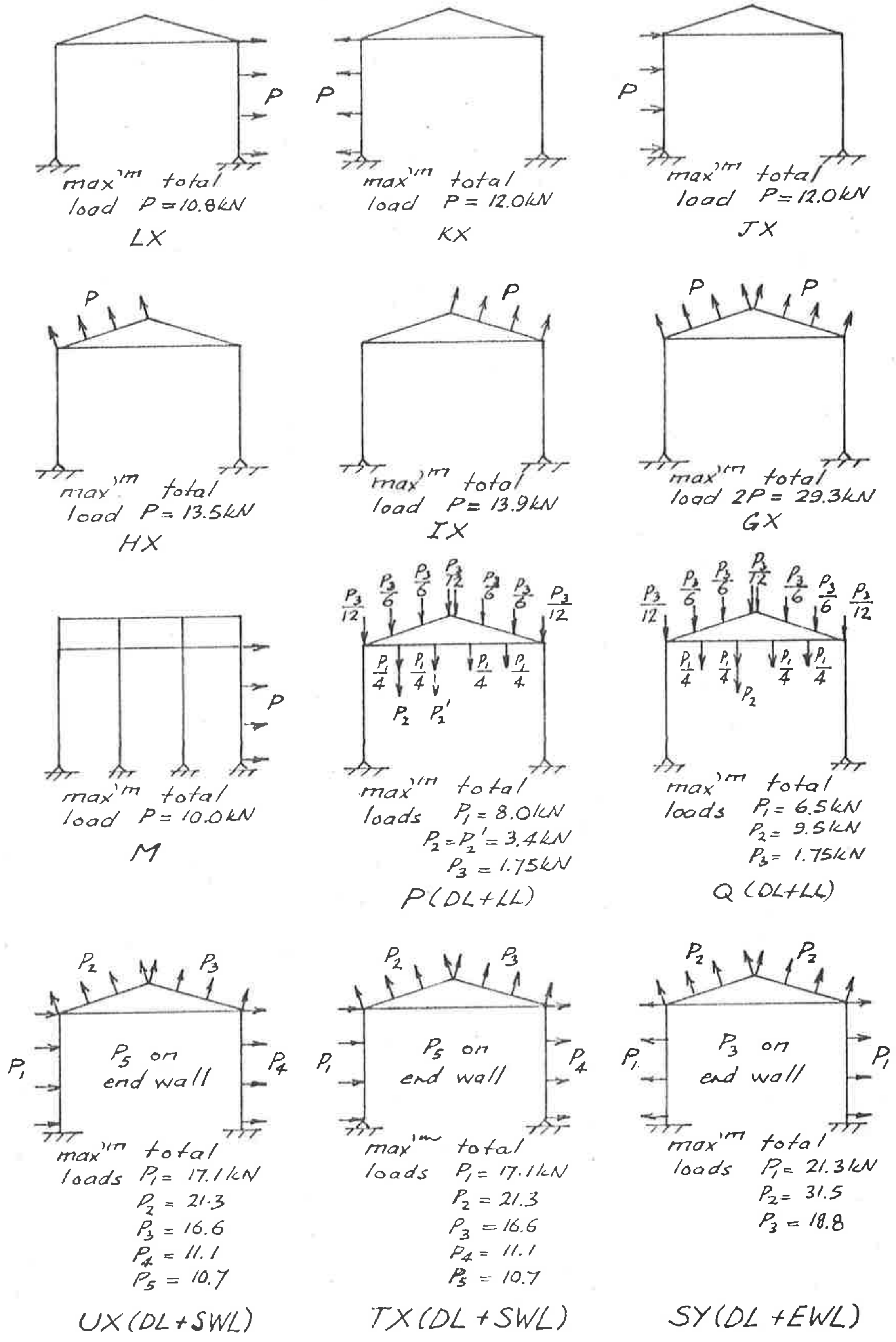
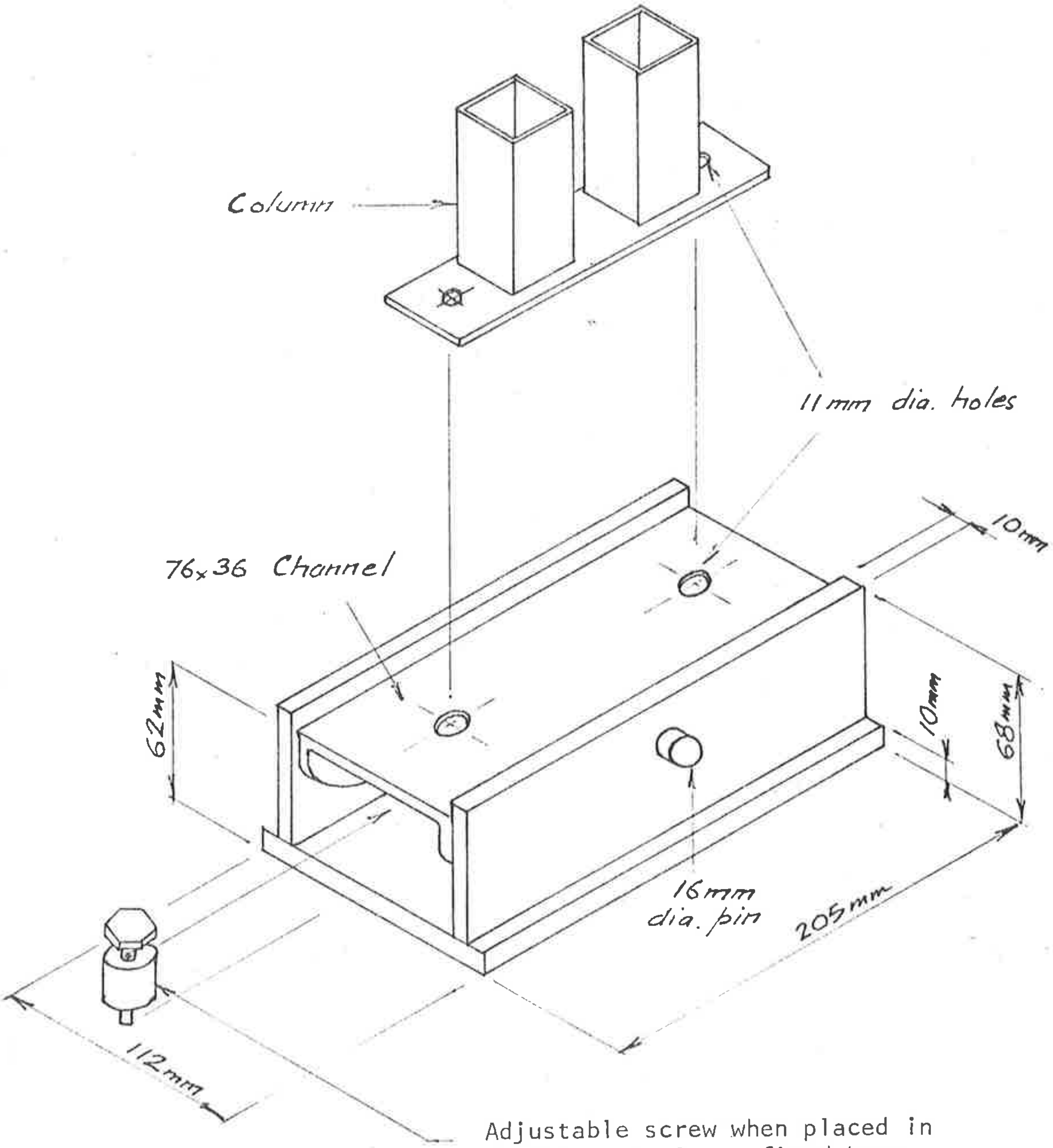

P
max'm total load P = 10.8kN
LX
P
max'm total load P = 12.0kN
KX
P
max'm total load P = 12.0kN
JX
P
max'm total load P = 13.5kN
HX
P
max'm total load P = 13.9kN
IX
P P
max'm total load 2P = 29.3kN
GX
P
max'm total load P = 10.0kN
M
P3/12 P3/6 P3/6 P3/12 P3/6 P3/6 P3/12
P1/4 P1/4 P1/4 P1/4
P2 P2'
max'm total loads P1 = 8.0kN
P2 = P2' = 3.4kN
P3 = 1.75kN
P (DL+LL)
P3/12 P3/6 P3/6 P3/12 P3/6 P3/6 P3/12
P1/4 P1/4 P1/4 P1/4
P2
max'm total loads P1 = 6.5kN
P2 = 9.5kN
P3 = 1.75kN
Q (DL+LL)
P2 P3
P1 P4
P5 on end wall
max'm total loads P1 = 17.1kN
P2 = 21.3
P3 = 16.6
P4 = 11.1
P5 = 10.7
UX (DL+SWL)
P2 P3
P1 P4
P5 on end wall
max'm total loads P1 = 17.1kN
P2 = 21.3
P3 = 16.6
P4 = 11.1
P5 = 10.7
TX (DL+SWL)
P2 P2
P1 P1
P3 on end wall
max'm total loads P1 = 21.3kN
P2 = 31.5
P3 = 18.8
SY (DL+EWL)

Figure 5.1



Adjustable screw when placed in position simulates fixed base conditions. One screw at each end.

Isometric View of Frame Base

Figure 5.2

These bases were bolted to a 200 UB which in turn was secured to the Strong Floor in the Civil Engineering Laboratory with three 30mm diameter bolts in the proximity of the bases and at midspan.

5.5 Loading

5.5.1 Equipment List:

As for Section 3.7.1. plus 150mm dia. pulleys

5.5.2 Load Mechanism

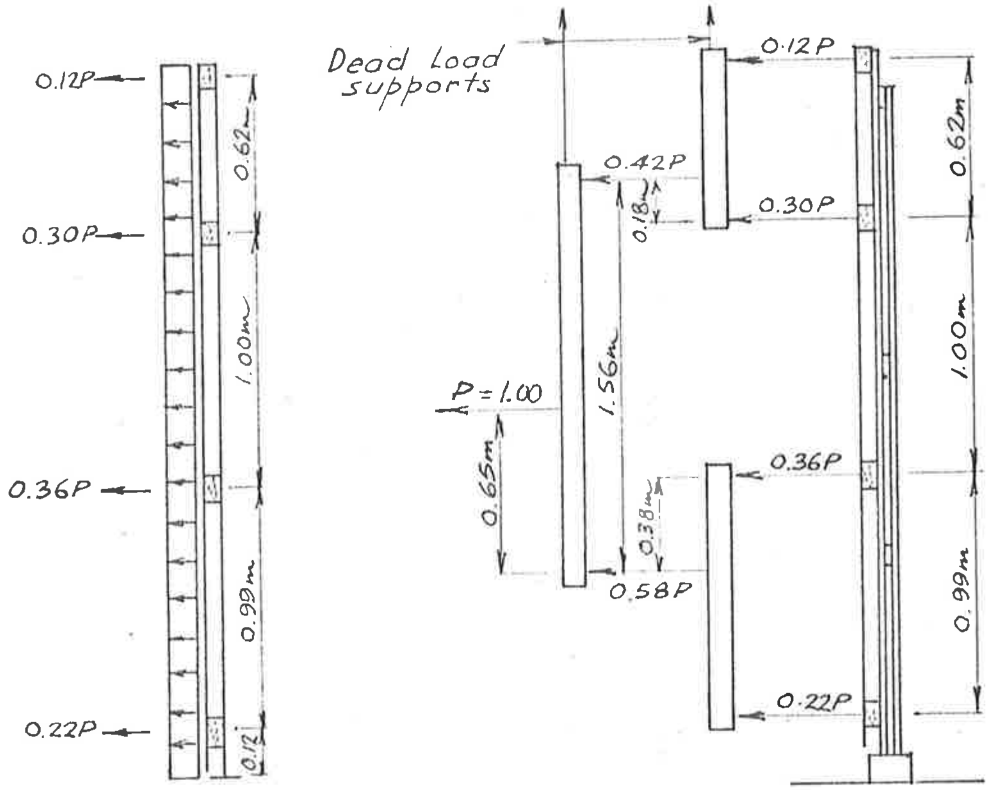
The loading mechanism for each frame is shown diagrammatically (see Figures 5.3, 5.4, 5.5).

These loading mechanisms were basically the same as for the Bare Frame tests. These loading mechanisms were then further extended by linking frames together so that only one load, therefore one jack, was necessary to load each of the five components which were two roof halves, 2 side walls, plus one end wall. (See Figures 5.6, 5.7, 5.8.)

The garage, after assembly was secured to the Strong Floor via 200 UB beams which acted as supported bases. Two large portal framed structures, consisting of 760 UB were erected parallel to one another, and over the garage in a transverse direction. The frames were then secured to the laboratory wall and the Strong Floor.

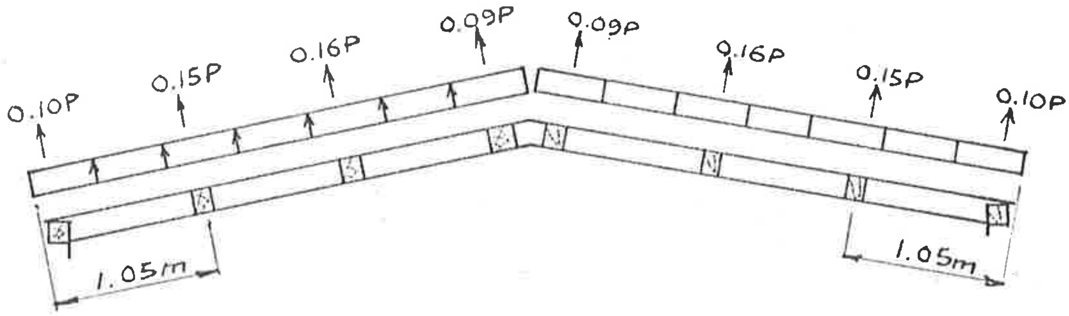
The loading mechanism used to transmit the load to the roof was accomplished via a pulley system connected to the portal frames.

Due to the low range in which the jacks were operated and the friction losses within the pulley system it was necessary to make load cells which were used to monitor the actual applied loads. These load cells were placed

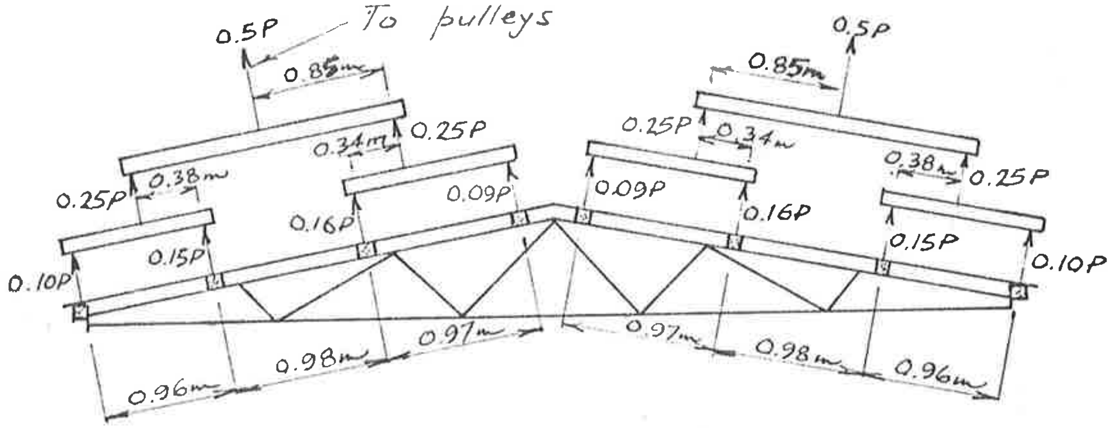


Side Load Details/Column

Figure 5.3

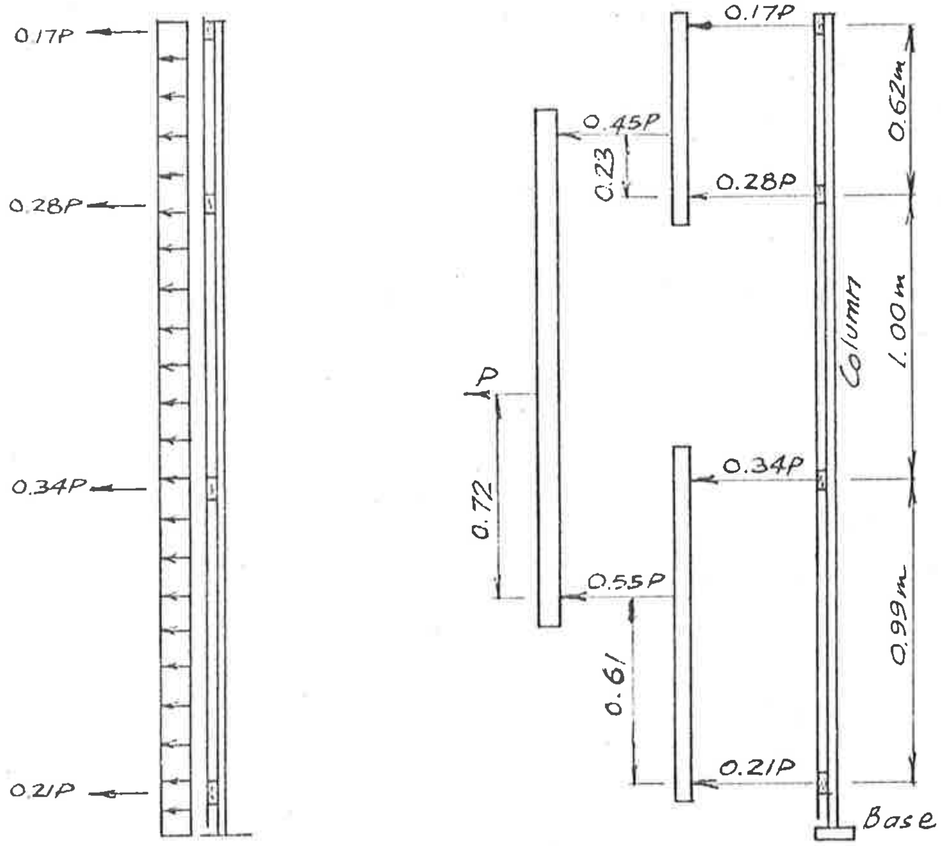


Equivalent U.D.L. on Purlins



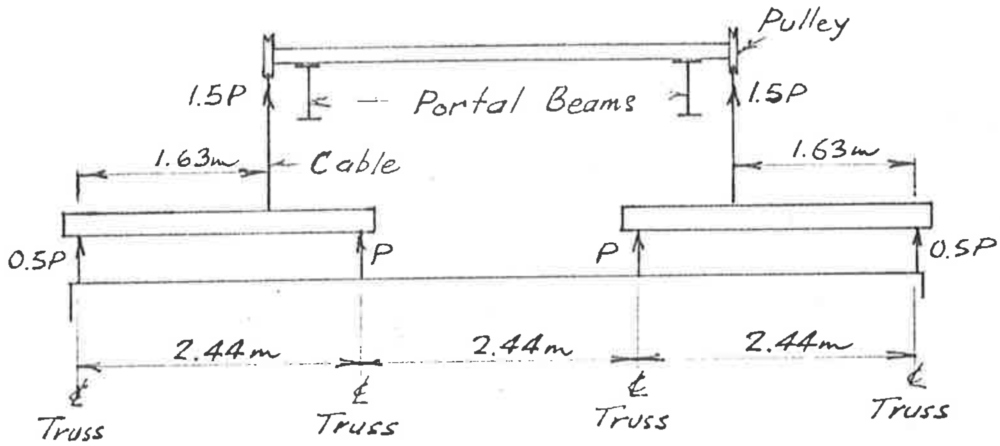
Vertical Load Details/Truss

Figure 5.4



Back Wall Load Details/Column

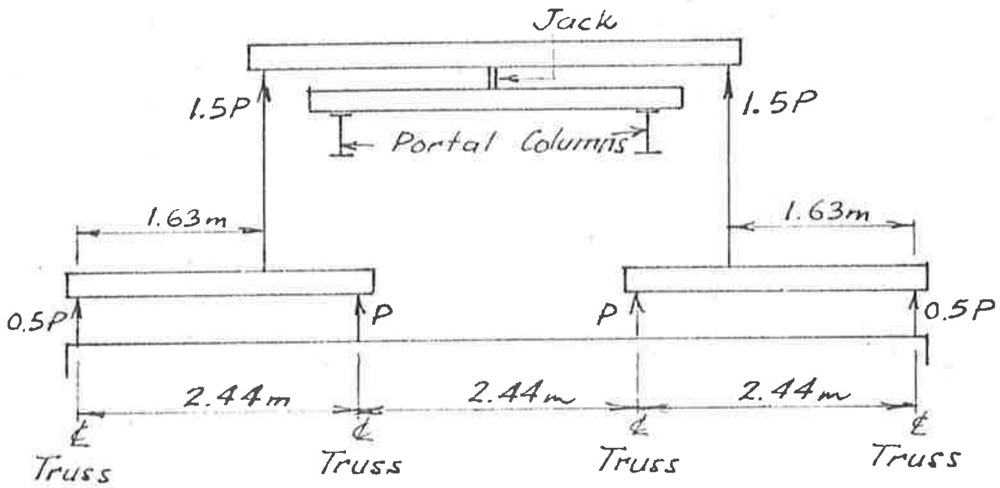
Figure 5.5



Note: Cables around pulleys and down to floor beams and jack.

Vertical Load Details/½ Roof

Figure 5.6

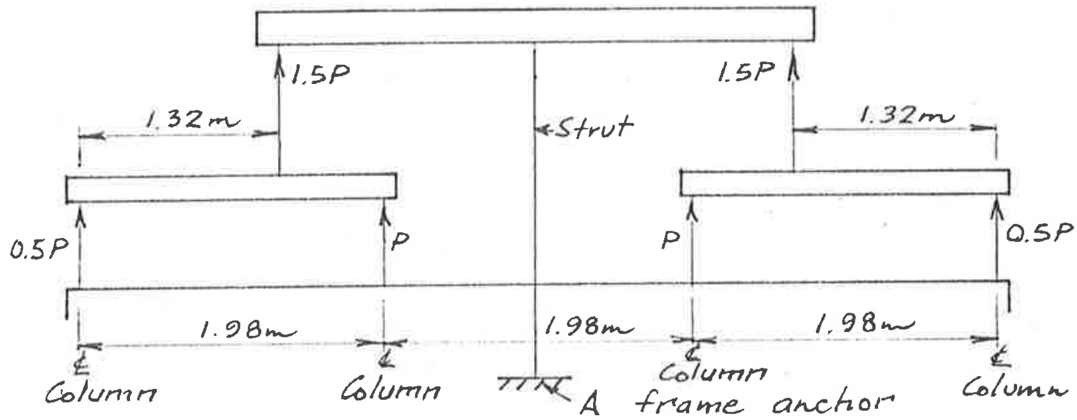


PLAN

Load Details/Side

Note: Other side jacked against building columns instead of portal columns.

Figure 5.7



PLAN

End Wall, Load Details

Figure 5.8

as close as possible to the garage and certainly between the end pulleys and the garage structure.

The total upward load was applied to a composite beam which provided a horizontal component, giving a resultant which simulated a normal uniform load on the roof.

The weight of the loading mechanism was compensated for by zeroing the load cells whilst they were carrying the weight of the jacking beams. This means that zero load for gauges was actually 1.0 times Dead Load. The backwall loading cases were carried out independently of any other load cases. The applied loads were transferred from a hydraulic jack through an independent loading mechanism to the back wall. For the backwall loading mechanism see Figure 5.5.

5.5.3 Load Cells

An accurate means of monitoring the total applied loads and at the same time a convenient method of recording them was required.

Three different load cells were used, but they were all basically steel rectangular bars of known cross-sectional area, wired with strain gauges on either side and connected to the scanning unit. Thus, each load cell occupied two channels in the data-logger system, making it possible to monitor each load cell with the digital volt-meter. Strains and hence load values were recorded in the same manner as all other strain gauges with the Facit out-put paper tape, and type printer.

The three different load cells were calibrated by test machine and a Philip's Bridge.

5.5.4 Hydraulic Jacks

The Enerpac pump and Plessey hydraulic jacks were used to apply the total loads.

For the end-wall load the jack was mounted against a rigid triangular frame which was bolted to the Strong Floor. For the upward load the jacks were secured to a supporting frame straddling the roof loading beams against which they were jacked.

Losses such as ram friction, hydraulic losses and deflection of the jack supporting bracket were avoided in both types of loadings by measuring the load as applied at the loading mechanism and not at the pump.

5.6 Instrumentation

5.6.1 Equipment List

As for Bare Frame tests see Section 3.8.1.

5.6.2 Strain Gauges

The strain gauges were secured to the frame members with Eastman 910 adhesive on the two opposing faces, top and bottom, on the centre-line of the longitudinal axis. Care was taken to ensure that even pressures were applied to the gauges as the glue set to avoid "hot spots" caused by uneven fixity, which could have resulted in inaccuracies of strain measurement. This uniform pressure was achieved with small magnetic clamps.

Frames were numbered 1 to 4 starting from the open front of the garage so Frames 2 and 3 were the internal frames with Frame 2 being the one nearest the open front.

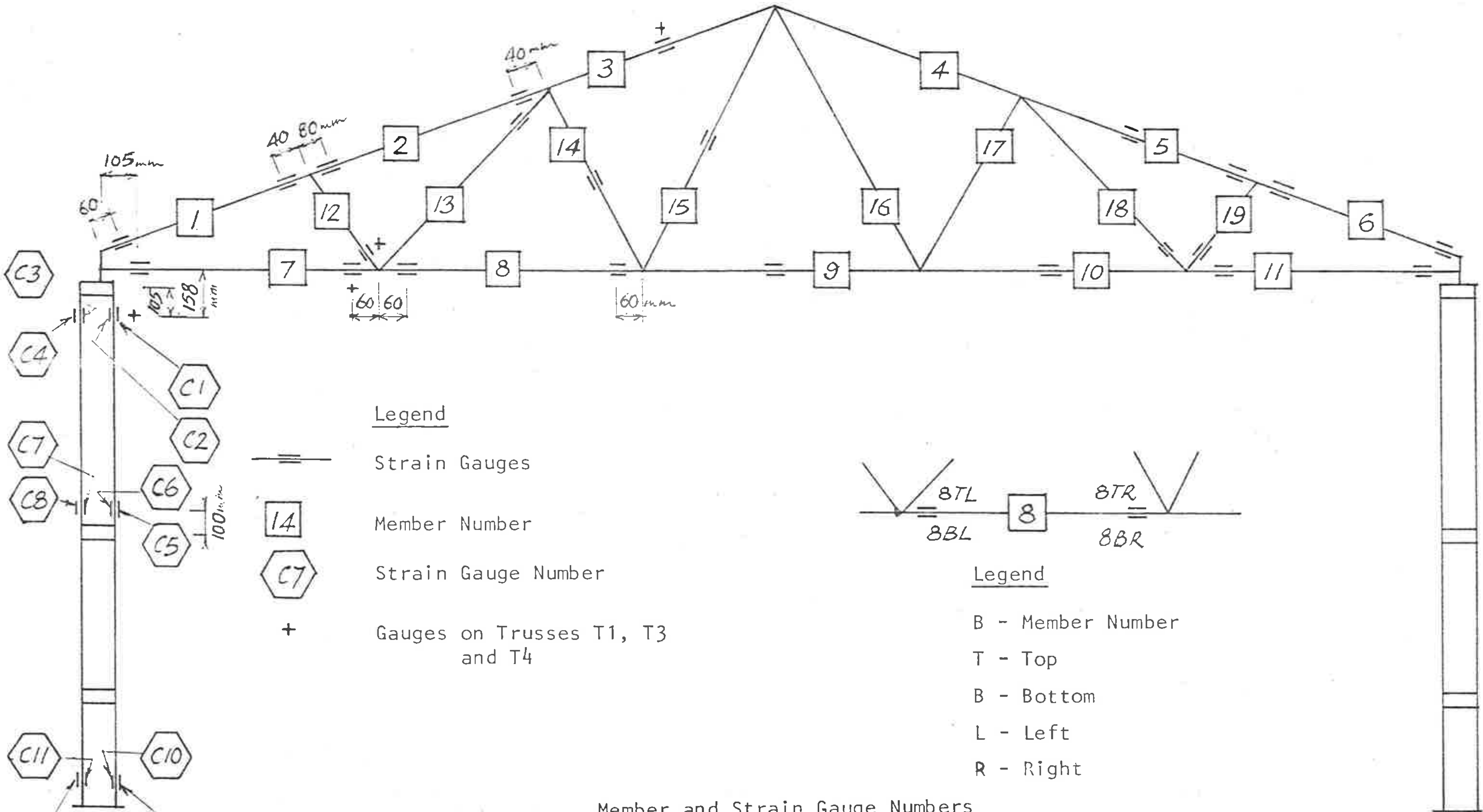
The strain gauges were concentrated about the truss-column joint of frame number 2, since this was the critical area. Other gauges were placed on the remainder of frame No. 2 in positions which could be used to monitor the general frame behaviour. Frame No. 2 was selected since this was more highly stressed than the other frames. Gauges were also placed on selected critical points of the other frames. For position of gauges see Figure 5.9.

In total, eighty-eight strain gauges were dispersed over the frames in such a manner that bending moments and axial forces could be found for the main members of interest. In order to avoid local stress concentrations, the strain gauges were placed approximately three member depths from the node points.

5.6.3 Circuitry for Strain Gauges

The strain gauge whiskers were soldered to a strain gauge terminal strip to prevent the accidental removal of the gauge. From this terminal strip two wires connected the strain gauge to the main terminal board. See Figure 5.10.

From the terminal board each gauge had its own active lead to a channel on the Scanner Unit. All gauges were connected to the same common dummy gauge and had a common connecting them at the terminal board. From this common a single lead connected this group to the B terminals on the Scanner. Therefore, all channels were connected by common links between their B terminals.



Member and Strain Gauge Numbers

Figure 5.9

Basically, the common dummy was connected to any one of the B terminals (which were linked together) and to the corresponding position of column A or B in the bottom row of connectors (both were internally joined).

5.6.4 Using the Logger System

The system was very flexible and could be used in many ways. For these tests it was used as follows:-

(a) Facilities - advantage in using this system was that many readings could be taken in a short period without having to manually record the values. The output can be in the form of a combination of the following methods:

(i) visual display of strain on digital voltmeter

(ii) visual display of strain and channel number on logger unit

(iii) punch data on continuous tape from Facit computer tape recorder

(iv) printed values using Facit printer.

It should be noted that when using the paper tape it was necessary to generate a row of feeder holes immediately prior to taking readings. This facilitated the reading of the tape with the PDP8/E computer tape reader, and storage on disc file in the computing centre.

(b) Using Logger - the data-logger was calibrated for a full scale range of 10,000 micro-strain by firstly balancing to zero each channel connected to a gauge and then setting the scale calibration to

10,000. This adjustment was controlled by two independent potentiometers. Each gauge was balanced against a dummy gauge which was not subjected to a load at any time, but did have approximately the same resistance as that of an active strain gauge. Once calibrated correctly the logger read micro-strain directly.

The logger can be programmed to monitor a range of channels or an individual one, with either a time controlled interval or manual start. As a series of channels is scanned the readings can be produced on any of the output devices.

The setting controls used for this series of tests to monitor 100 strain gauges connected by a 1/4 bridge common dummy were:-

11 mA gauge supply

100 mV range

10 readings per second

Primary program, group A

Manual Start, group A

Recorders 1 and 2

Channel identity

A more comprehensive guide can be found in the Logger, Scanner and Printer Operating Manuals.

(c) Logger Sensitivity

(i) Sensitivity: Due to the nature of the machine and the very small currents used the readings may flucuate with small resistance changes. The resistance variation may be due to surrounding conditions or poor connections. To restrict the temperature variations due to drafts, the strain gauges were protected with light felt pads.

In the measurement situation it is all too easy to have interference present which is passed to the voltmeter together with the wanted signal. Equipment such as the data logger can measure a few micro-volts with precision so that interference of hundreds of micro-volts (or even mV) may cause considerable error.

It was found during initial testing that marked interference with the results was caused by the operation of the overhead crane above the Strong Floor. Thus, the overhead crane was not used during testing periods.

(ii) Range of Potentiometers: Due to previous problems with the small resistance variation range for each channel within the data logger, (0.5 ohms), it was found necessary to alter the internal linking arrangements to increase the resistance variation range to 2.4 ohms. With this increased range it was possible to zero all the potentiometers with only one dummy gauge.

(iii) Wire Resistance: To ensure that the resistance variation between strain gauge leads was kept within the limits of variation permitted by the data logger, it was necessary to determine the resistance of the wire and it was 0.010 ohms/metre.

(iv) Accuracy: Due to slight variations in temperature resistance and machine limitations it was accepted that although the digital voltmeter could be read to six figures the last figure was subject to doubt. This could be seen in the instability of the last digit when monitored continuously.

No matter which scanning rate is selected the reading rate is restricted to the output of the recording devices. These were found to be:-

Paper tape	approx. $2\frac{1}{2}$ readings/sec.
Type printer	" 3 " "

This printing delay could be overcome by using a buffer store, but the instability problem would still remain.

(d) Fault Diagnosis

Applicable to 1/4 bridge, common dummy circuit.

The main problems arose due to -

1. Component Failure - gold contacts within units
   - faulty potentiometers
   - faulty strain gauge modules
   - faulty strain gauges
2. Resistances - Soldered joint and screwed terminals

The gold contacts between modules and panels of the Solatron Data-Logger system proved to be most troublesome. Dirty or faulty gold contacts caused many malfunctions requiring removal of front control panels, strain gauge modules, and input connectors, to be cleaned. These gold contacts were cleaned with a soft eraser and CRC 2-26 pressure pack aerosol solvent.

5.7 Testing Procedure

Prior to the commencement of load testing the Data Logger units were switched on to allow sufficient time for "warm-up" to occur before balancing and calibrating of strain gauge potentiometers. This "warm-up" period was necessary to avoid instability in recorded readings which became evident in early trial testing.

5.7.1 Side Wall Loadings (see Fig. 5.1, JX, KX, LX)

Three separate load cases were examined, all with pinned bases.

Due to the positioning of the strain gauges on the garage framework (refer to Fig. 5.9) it was necessary to apply individually the same inward and outward load cases to one wall and an equal outward load on the opposite wall. This was necessary since most of the strain gauges were grouped at critical locations on one side of the garage. By applying a load to the less instrumented side of the garage, it was possible to determine the load effects on the opposite side of the garage and hence evaluate what degree of load shedding occurred through the cladding.

Load increments of approximately 5 kN were applied with a hydraulic jack and the strain readings recorded. In addition to the strain readings the deflection of the garage in both the lateral and longitudinal directions was measured.

5.7.2 Roof Uplift Loadings (see Fig. 5.1, GX, HX, IX)

Both roof planes were loaded independently and then simultaneously for the pinned based condition.

Load increments of approximately 5 kN were applied with hydraulic jacks, and the strain readings, along with lateral and longitudinal deflections were recorded.

5.7.3 End Wall Loadings (see Fig. 5.1, M)

The end wall was examined with pinned bases. Load increments of approximately 2.5 kN were applied with a hydraulic jack and the lateral deflection of the end wall in conjunction with longitudinal deflection of the garage at eaves level was measured.

5.7.4 Combination of Dead and Live Loads (see Fig. 5.1, P, Q)

Two separate load cases were examined, both with pinned bases.

The Dead plus Live loads were simulated by the application of downward forces at the bottom chord node points via a load assembly and hydraulic jack. This was representative of dead plus maintenance load applied to the external roof area. An additional live load was applied separately to two bottom chord node points.

5.7.5 Combination of Side Wall, Roof Uplift and End Wall Loadings (see Fig. 5.1, SY, TX, UX)

Three separate load cases were examined. One set of loads gave two load cases as those loads were applied with both pinned and fixed bases, whilst the other load case was a different set of loads applied with a fixed base only.

These load cases simulated wind loading conditions, incorporating both external and internal wind loadings. The loads were transmitted to the structure via the loading assemblies and hydraulic jacks as used in previous cases.

Loading increments appropriate to the loaded area were applied in proportion to the maximum load applicable to that area.

In addition to the strain readings the deflection of the garage in both the lateral and longitudinal directions was measured.

5.8 Results

5.8.1. General

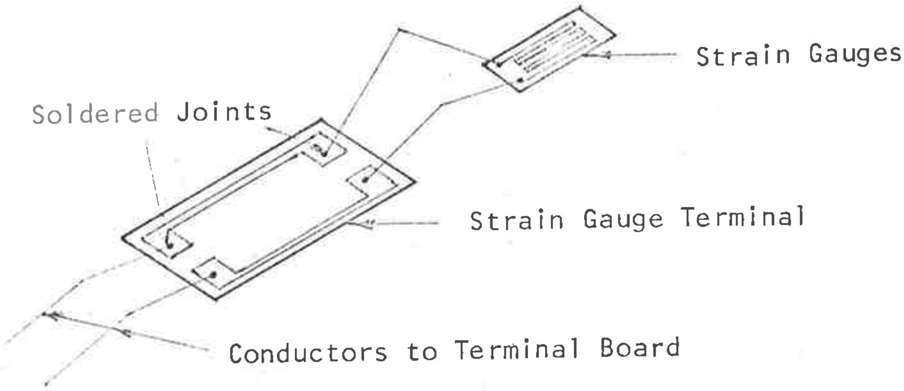
The test results are too numerous to be included in full either here or in the Appendix but are available for perusal.

The key results are used in Appendix D and will be mentioned there.

Results for the combined load cases (P, SY, TX, UX) for the maximum loads applied are listed in Appendix C.

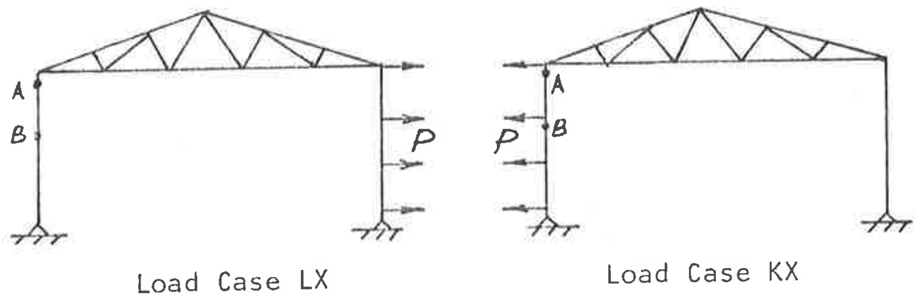
5.8.2 Effect of Cladding

Load cases KX and LX were compared to estimate the amount of load shedding through the cladding. These cases were used as they were of nearly equal and opposite magnitude, applied to each side wall in turn.
Bending moments and axial forces were compared for the column gauges located at A and B (see Fig. 5.11) which were placed on one column only.



Typical Strain Gauge and Strain Gauge Terminal

Figure 5.10



Location of Common Strain Gauges

(see Figure 5.9)

Figure 5.11

The columns were theoretically pinned top and bottom so the cladding should have carried all the lateral load thereby producing no stress in the column away from the applied load. The test results indicate that this was predominately so, but not completely, so that some small fixity at the top joint possibly did exist.

Another method used to assess the effect of the cladding was to compare the transverse deflections for the bare frame and those for the frames in the assembled garage, see table below.

		Lateral Load/ Frame (kN)	Deflections (mm)	
			Pinned Base	Fixed Base
Unclad frame		1	90	40
Assembled Garage TX, UX				
Frames	1	9.4	47(19)	49(17)
	2	"	43(18)	47(17)
	3	"	28(15)	40(18)
	4	"	19(8)	19(8)

Bracket figures are the residual deflections after unloading. Fixed base deflections were generally the greater of the two sets of deflections and this was due to residual building slackness resulting from the initial loading of the building using pinned bases.

Longitudinal deflections of the side walls were also measured and varied from 5 to 10mm at maximum load.

From the table it can be seen that the cladding produced a dramatic reduction in the deflections, thus indicating that the cladding carried the major part of the load.

5.8.3 Dead Load and Live Load Cases

Load cases P and Q were tested for domestic and industrial loads respectively. These loadings were in accordance with AS 1170, part 1 for dead and live loading requirements.

The external roof live load for load case P was representative of a normal maintenance live load with an occasional domestic point load suspended from the bottom chord of truss number two. This occasional load was alternatively placed at the first and second interior bottom panel points.

As the live loads were only applied to one truss they were calculated as the working loads multiplied by a load factor of two and these ultimate loads were then finally increased by 30% to allow for load shedding via purlins to adjacent trusses.

The garage supported this load.

For load case Q the external roof load was again a maintenance live load with an occasional industrial point load suspended from the second interior bottom chord panel point of truss number two. Upon application of this occasional load it was observed that failure occurred at the junction of the top and bottom chords of the truss by weld failure. The truss top and bottom chords were stressed to yield at several points. The loads applied were less than the full test loads required for this industrial building case but greater than those required for the domestic building case. The failed joint was re-welded so that further tests could be done.

5.8.4 Combined Wind Load Cases

Load cases SY, TX and UX were all subjected to a series of load increments. These load increments simulated wind loads in accordance with AS 1170, part 2 - 1975, using terrain categories 2 and 3, with a wind velocity of 42 m/sec. These loadings were applicable for the metropolitan area of Adelaide.

Strain readings were recorded for all cases under the following loading conditions:-

Load Case	Terrain Category	Internal Pressure	Load Factor
SWL - TX and UX	3	0.0	1.2
	3	0.5	1.2
	2	0.0	1.2
EWL - SY	3	0.5	1.2
	3	0.8	1.2
	2	0.0	1.2
	3	0.3	2.0
	3	0.8	2.0
	2	0.4	1.2

Load factors of 1.2 and 2.0 were used. The South Australian Building Act 1970-1976 specified a load factor of 1.2, whilst the S.A.A. Steel Structures Code AS 1250 - 1975 specified a load factor of 2.0.

Observations of progressive results showed that when the yield stress was reached at a particular point, any additional applied load was transferred to adjacent lower stressed members. This indicated progressive formation of plastic hinges. The garage failed at DL + EWL, SY under wind category 2 with an internal pressure of 0.4 and a load factor of 1.2.

Failure was due to both weld failure at the truss end and by bottom chord buckling.

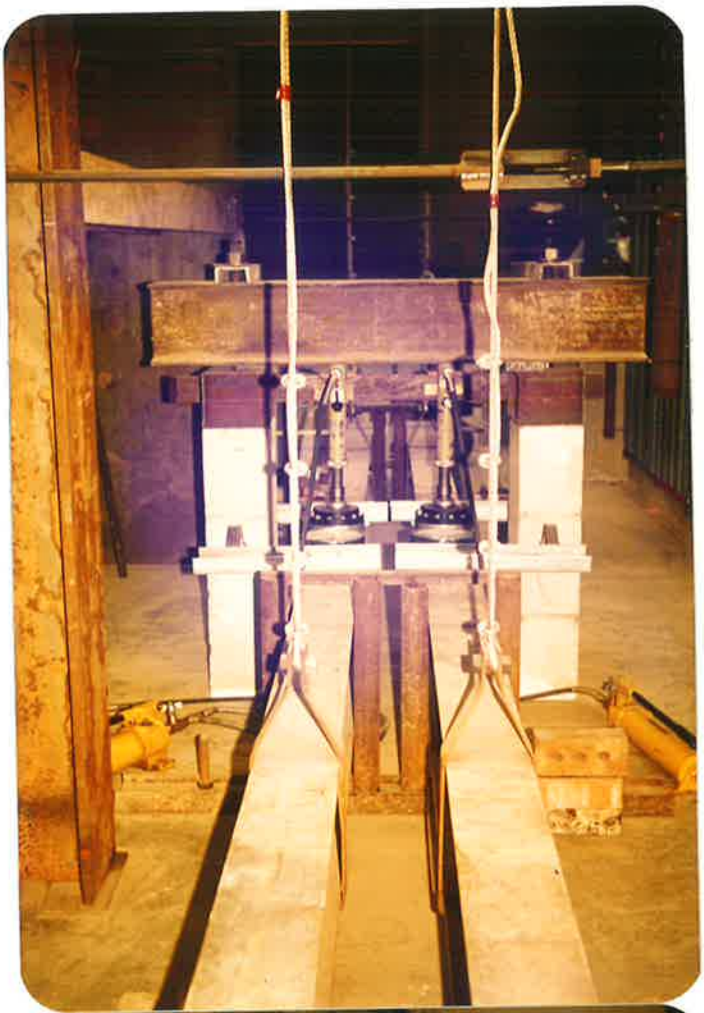
5.9 Summary

Extensive load tests were done and the results obtained. The tests showed that nearly all of the lateral load was taken by the sheeting and that resulted in end wind load governing over side wind load. Removal of the frame knee braces improved the frame strength.

The structure overall had the required strength to the S.A. Building Act 1970-76 for the vast majority of the Adelaide Metropolitan Area.

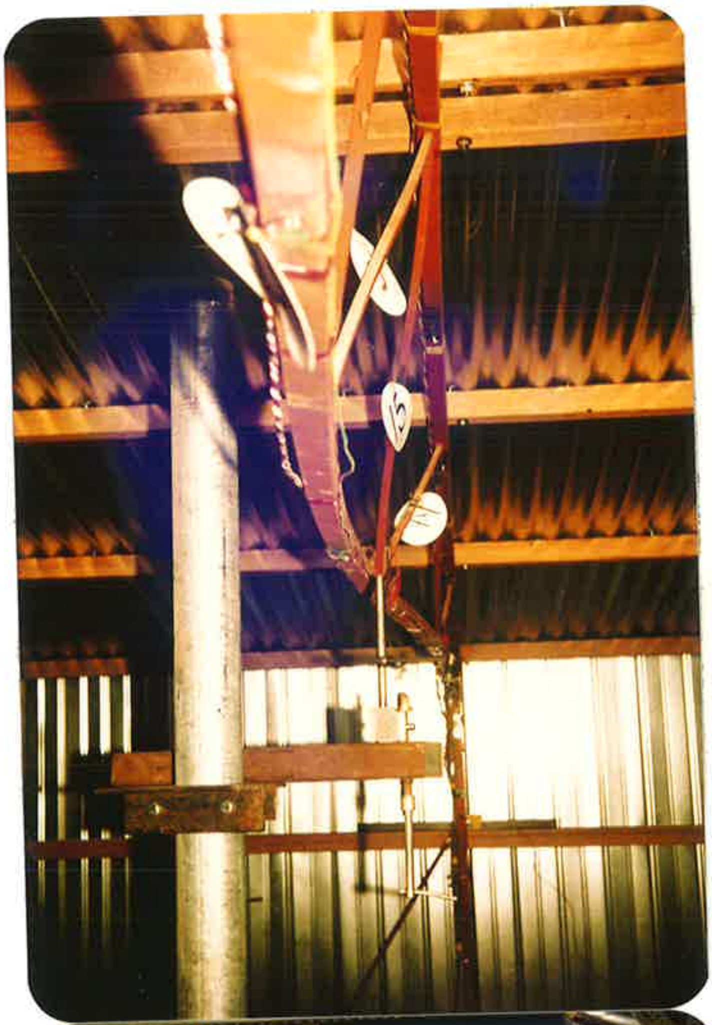
Roof Loading

Roof & Side Wall Loading



Roof Jacks

End Wall Loading



DL-EWL, T2

Data Logger

DL - EWL, T2

DL - EWL, T3

6. DISCUSSION OF EXPERIMENTAL AND THEORETICAL RESULTS

6.1 General

This section compares the test results as detailed in sections 3, 4 and 5 to the structural review as detailed in Appendix D. The structural review was basically in accordance with the design codes but was extended where necessary to give agreement with the strength as shown by testing. This discussion will illustrate those extensions.

In general, as expected, the actual strength determined from testing was greater than the strength determined from a structural analysis carried out strictly in accordance with the design codes. This was mainly due to the actual yield stress of the steel used for the building being much higher than the guaranteed minimum yield stress given in the steel code. The increased strength was also probably due partly to the necessary conservatism of the design codes.

The load cases considered were

(a) Dead Load plus Live Load (DL + LL) which examined the building under vertical downward loads.

(b) Dead Load plus End Wind Load (DL + EWL) which examined the trusses under uplift loads and the columns under lateral loads.
The load pattern was symmetrical here so no nett lateral load existed.

(c) Dead Load plus Side Wind Load (DL + SWL) which basically examined the building under lateral loads.

The truss to column joint had a large eccentricity and that resulted in the end panels of the truss behaving as a rigid frame instead of as a truss. It was found that a plastic theory analysis of the truss end panels was necessary in order to obtain the highest possible analysis load capacity. The end panels of the trusses governed the truss strength due to the high moments there (caused by the eccentricity) adding to the large axial forces also present. The actual behavious of the truss end panels was confirmed by comparing the analysis results with the test results (detailed in Appendix D, Parts D3.1, D3.2 and D3.3).

The columns behaved as Vierendeel girders and so the chords had high bending moments as well as considerable axial loads. The top panel was the critical panel due to its longer length than the other two panels. The actual behaviour of the column was confirmed initially by a single bare column, load test (see Figure 3.3) and later by comparing the analysis results with the assembled garage test results (detailed in section D5).

Some load transfer by the purlins from the internal trusses to the lighter loaded end trusses was possible because

(a) The purlins were in one single length over the total garage length.

(b) All trusses were identical and so of equal strength.

The structural review used this load transfer in both the DL + LL and particularly the DL + EWL load cases.

6.2 DL + LL

6.2.1 General

Only vertical loads were applicable here and therefore due to the truss-column framing style of the building the sheeting was unable to act as a shear diaphragm and thereby support some of the load. The discussion here then was concerned only with the strength of the purlins, trusses and columns.

The building was tested under load cases P and Q (Figure 5.1) and the results were reported in Appendix C1 and Section 5.8.3. Load case P covered dead load plus roof maintenance live load plus 1.3 kN concentrated live load. Load case Q was basically an attempt to increase the concentrated live load to 4.5 kN to satisfy the requirements for an industrial building.

Only one truss, namely the first internal truss from the open front of the garage (T2), was proof tested. The loads applied to T2 were increased to compensate for load shedding by the purlins to the adjacent trusses. However, the amount of load shedding was greater than anticipated. (Appendix C1).

6.2.2 Purlins

These did not fail under testing and were shown by analysis (Section D1) that they almost satisfied the timber code AS1720 (Reference 13) when the live load was only allowed to be present for six hours. Reducing the live load duration to one hour would enable the use of a 1.6 load duration factor (1.5 for 6 hour loading) and then F17 grade fully satisfies the timber code.

6.2.3 Trusses

The trusses were unable to support the test load Q but were reasonably proven to support the test load P. Due to more than anticipated load shedding from the loaded truss the full required load for a domestic building plus 1.3 kN concentrated load was not in fact applied and that was why it cannot be absolutely stated that load case P was supported by the truss.

A plastic theory analysis in accordance with the steel codes AS1250, AS1538 (References 14, 15) was carried out and that is detailed in Appendix D3. The results of that analysis are as follows:

(a) The truss alone (i.e. without any load shedding via the purlins) was just able to support the domestic load case of dead load plus roof maintenance live load. The required yield strength was 250 MPa.

(b) The addition of a concentrated live load of 1.3 kN to the bottom chord and placed at the worst location namely a panel point adjacent to a column, overstressed the end panel of the truss, adjacent to the 1.3 kN load. The yield strength used was 250 MPa. Both chords of the end panel formed plastic hinges at both ends thereby producing collapse short of supporting the required load. This fact was supported by the load test. Testing (Appendix A1) showed that the yield stress could be taken as high as 360 MPa. If the yield stress was taken as 320 MPa then the truss alone supported the total loads of dead load plus maintenance live load plus 1.3 kN concentrated live load.

On considering load transfer via the purlins from the internal to end trusses it was shown (Appendix D3) that the purlins could transfer 11% of the load. To support the remaining 89% of the load the required yield strength was 280 Mpa.

It was also pointed out that it was only the chords of the end panel that were critically stressed i.e. the rest of the truss chords and in particular the internal members were never critically stressed.

6.2.4 Columns

The analysis of the columns supported the test results in showing that the columns were never critically stressed. They were in fact perfectly safe even with F_y = 250 MPa.

6.2.5 Comparison, Test to Analysis for the Trusses

Overall then the analysis supported the load test and showed that the truss supported the loads of Case P either by

(a) Fy $\geqslant$ 320 MPa

(b) Fy $\geqslant$ 280 MPa and with 11% of the load transferred by the purlins from the internal trusses to the more lightly loaded end trusses.

This case was more likely than (a) as truss deflection forced the purlins to transfer load.

6.3 DL + EWL

6.3.1 General

As there was no nett lateral load applied from this load case so then as for DL + LL no shear loading of the sheeting resulted and so none of the applied load could be supported by the sheeting. This discussion then was only concerned with the purlins, girts, trusses and columns.

The maximum test load, at which the garage failed, was load case SY (Figure 5.1 and Appendix C2) which was determined from Location Adelaide, Wind Category 2, Internal Pressure Coefficient 0.4 and a Load Factor of 1.2 WL - DL.

6.3.2 Purlins and Girts

These did not fail under testing and the analysis (D1) showed that under the maximum test loads F17 grade timber fully satisfied the timber code AS1720 (Reference 13).

6.3.3 Trusses - Analysis

The plastic theory method of analysis was again applied in an attempt to justify the trusses by analysis as well as by testing. As the loading was symmetrical both end panels of the truss were theoretically equally stressed. Even with the end panels fully stressed (at state of collapse with four plastic hinges) the trusses could only support the test load by using Fy = 360 MPa and transferring 17% of the load via the purlins from the internal trusses to the lighter loaded end trusses.
The purlins were shown capable of transferring 17% of the load and the use of Fy = 360 MPa also was confirmed (Appendix A). Appendix D4.1 gives the details of the analysis.

Another problem was the buckling of the bottom chord as this member was in compression under DL + EWL.
There was no longitudinal bracing anywhere along the bottom chord. Therefore any lateral support actually afforded to the bottom chord could only have been provided by the truss diagonals (acting as cantilevers). The strength of the bottom chord under transverse buckling was determined as

(a) Braced at the ends only, then L = 5.94 m, PAC = 1.2 kN

(b) Braced at all four internal truss panel points, then L = 1.2 m, PAC = 35.5 kN

(c) Braced at the truss panel points adjacent to the columns but unbraced at the two internal panel points, then L = 3.6 m, PAC = 6.5 kN

Note: The truss end panel was sway unrestrained for vertical buckling so there PAC = 21.3 kN but all other truss panels were sway restrained for vertical buckling and so there PAC = 35.5 kN.

The actual mean force that produced transverse buckling of the central section of the bottom chord was 14.5 kN. It was then hypothesised that the true lateral restraint condition lay between cases (b) and (c).

An investigation was then done to confirm or otherwise that the diagonals did in fact provide effective lateral support to the bottom chord. (As the diagonals had to act as cantilevers to provide this support it followed that the top chord or purlins or both had in turn to support the end moment and transverse force from the diagonals). The diagonals did in fact (as shown by the analysis), provide considerable lateral restraint to the bottom chord. The lateral restraint seemed sufficient to provide full restraint at the outer panel points and partial restraint at the inner panel points. That information confirmed the hypothesis that the load required to cause transverse buckling of the centre panel of the bottom chord would lie between 6.5 kN and 35.5 kN and quite likely would be the observed failure load of 14.5 kN.
The analysis is detailed in Appendix D4.2.

6.3.4 Trusses - Comparison, Test and Analysis

On examining the test results (Appendix C2) it was observed that the analysis hypothesis, namely that the end panels at both ends of the truss were fully stressed at the maximum test loading, was supported.

The other analysis hypothesis was concerned with bottom chord buckling. From the test results and a subsequent buckling analysis it was hypothesised that the truss diagonals provided effective lateral restraint at the outer panel points (closest to the columns) and partial lateral restraint at the inner panel points. The hypothesis was then supported by calculating the required restraint stiffness and comparing it with the actual stiffness as provided by the truss diagonals (acting as cantilevers). The analysis did not absolutely prove that the diagonals provided the required restraint stiffness and so the only absolute strength justification came from the load test.

A common buckling analysis of such a member would have regarded the member as laterally unrestrained over the whole length of 5.94 m. This thesis has clearly shown that such an approach would produce a very conservative result. Therein that approach produced a strength capacity of 1.2 kN. So the diagonals provided sufficient lateral restraint to raise the buckling strength from 1.2 kN to 14.5 kN. However, the panel point restraints were insufficiently stiff to raise the buckling strength to the maximum possible value of 35.5 kN.

6.3.5 Columns - Analysis

This analysis was done initially for both pinned and fixed bases but always for pinned heads. (Appendix D5). The base plate was shown to be very weak and certainly unable to support the fixed base condition and that therefore necessitated discarding the fixed base.

Even using the pinned base it was necessary to use plastic theory to avoid overstressing the base plate. As the whole column acted as a Vierendeel girder plastic theory was clearly the preferred method of analysis.

Load transfer via the girts from the internal columns to the end columns was investigated but discarded due to lack of strength plus extreme flexibility of the girts. Composite action between the sheeting and the column was also investigated but also proved of negligible assistance and so was also discarded. The analysis therefore indicated that the column alone supported the total load.

The analysis showed that the columns safely sustained the test load provided that Fy = 280 MPa and also that the top panel (nearest the truss) was the critical panel.

6.3.6 Columns - Comparison, Test and Analysis

The test results (Figure D14) clearly showed no reversal of axial load from the top to the base of the column thus supporting the hypothesis of pinned bases. The columns certainly did not fail under test and the analysis confirmed the column strengths. However, the test indicated lower stresses than determined by analysis. Load sharing was investigated (6.3.5) but seemed to offer negligible assistance.

Overall the column was less severely stressed than the truss. The yield stress only needed to be increased to 280 MPa for a satisfactory analysis whereas 360 MPa plus 17% load shedding via the purlins was needed for a satisfactory analysis of the truss.

6.4 DL + SWL

6.4.1 General

This was the only load case that had a nett lateral load and so was the only case that called upon the roof sheeting to act as a shear diaphragm and so support some or all of the lateral load. Except for this lateral load effect this case was less severe than DL + EWL. Therefore the discussion here was only concerned with the sheeting acting as a shear diaphragm. The maximum test load was load case TX (Figure 5.1) which was determined from Location Adelaide, Wind Category 2, Internal Pressure = Internal Suction = 0 and a load factor 1.2. The total applied lateral load was 13.3 kN of which 2.2 kN was applied directly to the back wall. The balance of 13.3 - 2.2 = 11.1 kN was then supported by the roof sheeting plus the steel frames.

6.4.2 Roof Sheeting - Analysis

The Garage was initially analysed as a four pinned frame with the pin joints at the tops and bottoms of the columns. That meant that the roof sheeting had to support the total lateral load of 11.1 kN. The column bases were true pin joints but the column to truss joint was assumed as a pin joint only after observing that the ratio of base frame deflection to clad frame deflection was 0.05. That ratio indicated that the frame alone had negligible resistance to a lateral load so resulting in the pin joint assumption.

Then basing the Garage roof strength on the results of testing of one isolated roof panel gave a roof strength of 2 x 4.0 = 8.0 kN. (Section 4.5, Appendix D6.2) Thus a gap of 11.1 - 8.0 = 3.1 kN remained unexplained.

Now the test panel deflected 7.0 mm at 4.0 kN shear load (Figure 4.12) whereas the differential deflection between the end wall truss and the adjacent internal truss was 9.0 mm (Figure C3b). That fact could be taken as indicating that the roof was at its ultimate and that some of the load was in fact supported by the frame. Against this however, was the fact that observations of the garage at maximum load certainly did not indicate that the roof sheeting had failed. Nevertheless the assumption of a pin joint at the truss to column connection was revised. On treating the top joint as a possible moment connection it was found (D6.3) that the frames could possibly have supported 1.7 kN. Thus still leaving a gap of 3.1 - 1.7 = 1.4 kN.

That then suggested that either the test panel gave a conservative result or that somehow the roof configuration gave greater strength than indicated by the test panel. Surely it can be stated that the sheeting supported up to 8.0 kN lateral load. So under the maximum applied lateral load of 11.1 kN the sheeting supported 8.0 kN i.e. 72% of the total.

6.4.3 Roof Sheeting, Hypothesis for Total Load Support

The following hypothesis cannot be proven or otherwise without additional testing but is put forward as an attempt to justify the hypothesis that the roof sheeting did in fact support all of the lateral loads.

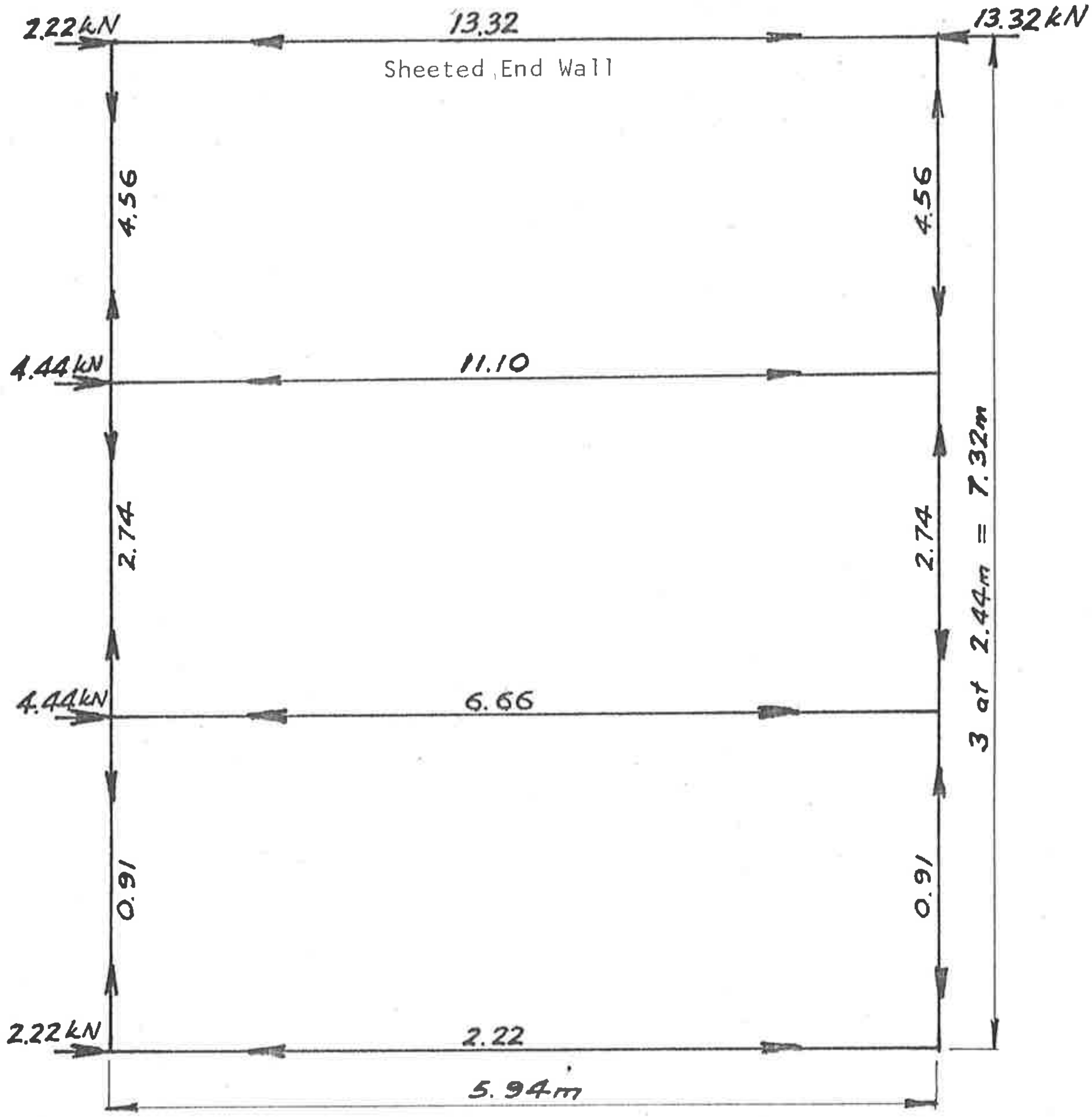
(a) The assembled garage had to transfer a maximum shear of 4.44 kN (Figure 6.1) from any one truss via the purlins to the sheeting whereas the tested roof panel had to transfer $4 \times 2 = 8.0$ kN from the end beam via the purlins to the sheeting. Therefore from that effect the loads on the fasteners immediately adjacent to the trusses were less than those for the fasteners immediately adjacent to the end beam, for the test panel.

(b) Comparing the loads on each of the 12 edge fasteners (per 2.44 m bay) caused by complementary shear the following was obtained:

$$\text{Test Panel force} = \frac{8.0 \times 2.44}{6 \times 12} = 0.27 \text{ kN}$$

$$\text{Assembled Garage force} = \frac{11.1 \times 2.44}{5.94 \times 12} = 0.38 \text{ kN}$$

The ultimate strength for an individual fastener was 0.38 kN (4.4.1), this then suggested that the edge fasteners were fully used in resisting the complementary shear. That means that the garage was weakened as regards to the test panel but the nett loss was $(0.38 - 0.27)/0.38 = 0.30$ of a fastener along each perpendicular



Roof Plan

Figure 6.1

(to the side walls) line of fasteners.

However the other case, namely (a), that also has a fastener strength of 0.38 kN had a reserve strength of (8-4.44)/8 = 0.45. Overall then, it seems reasonable that the two effects could be regarded as cancelling each other.

(c) The fastener strength at the laps for transferring load from sheet to sheet was determined as 2.0 kN (4.4.2). Assuming the worst case of those fasteners also transferring load from the purlins their strength would still be of the order of 2.0 - 0.38 = 1.62 kN.

As there were four purlins in each half of the roof that still results in an ultimate strength of 4 x 1.62 x 2 = 13.0 > 11.1 kN.

(d) If the actual garage supported more than 8.0 kN shear load then it was possible that sheet buckling and not sheet tearing (at the fasteners) would become the critical failure mode.

This was investigated using Easley's formula (reference 25) but it was found that the sheet buckling load was 26.0 kN > 11.1 kN and so sheet tearing remained the critical collapse mode.

So points (a) to (d) substantially justify the hypothesis that the roof sheeting did indeed support all of the lateral load. This can only remain a hypothesis for this thesis and in practice the ultimate shear capacity of the roof must be taken as 8.0 kN.

6.4.4 Side Wall Sheeting

The side wall sheeting was basically required to support the complementary shear force resulting from the normal shear force (on the roof) and transfer that force to the ground. The complementary shear load accompanying the shear load of 11.1 kN was 8.21 kN. The wall shear strength was 16.0 kN so the side walls easily accommodated these complementary shear forces. The side walls were then far lighter stressed under DL + SWL than was the roof. (D6.4).

6.4.5 End Wall Sheeting

That wall took the roof shear and transferred it to the ground. The end wall received load from all 3 bays whereas the roof only received shear loading from 2½ bays due to the last ½ bay loading being transferred from the side walls direct to the end wall instead of via the roof. Assuming that all the lateral loading was taken by that wall, it's total load was 13.3 kN. The actual shear strength of the end wall was shown in D6.5 to be 13.0 kN or 97.7% of the total load. As the load on the end wall was mainly applied in the same style as on the test panel the hypothesis used in 6.4.3 for extending the roof strength cannot reasonably be applied here. This wall did not however, under test, show any sign of imminent failure. It can be reasonably held that the end wall has been shown as capable of supporting almost all, if not all of the lateral load.

6.5 Applicability of Results

The test loads applied to the Garage have determined the structural characteristics of the particular tested Garage. The Garage has in fact, been Proof Tested. The

Proof Test does not necessarily prove the strength even of all identical Garages (i.e. same materials, dimensions and style) because only one Garage has been tested with the result that no strength confirmation exists. The structural review of Appendix D showed that the Garage only survived due to a steel yield stress of 360 MPa. It would be necessary to confirm that this high yield stress holds in general in order to use the test loading as indicating the common strength of these Garages.

The cladding has been shown by testing to be capable of supporting a shear load of 8.0 kN. Again, this figure also lacks confirmation so if used without any additional testing, could only be at the designer's responsibility.

If any building authority requires further verifictation of the Garage strength then the following additional tests are suggested:

(a) Roof and wall panels to verify or otherwise their shear capacity.

(b) Tensile testing of the steel to be used for the main chords to verify or otherwise their yield stress.

With this additional information, the Garage strength could be either verified or if necessary, have the strength adjusted.

7. CONCLUSIONS

Experimental and analytical investigations have been completed and these show the strength, including cladding, of one particular Domestic Garage.

Vertical loads were essentially carried by the steel framework but some load transfer to adjacent more lightly loaded frames by the purlins was noted. This was probably of the order of 17% for DL + EWL.

Horizontal loads at the eaves level were mainly taken by the sheeting as the steel frame itself had little resistance to this type of loading. The sheeting supported at least 72% of the maximum lateral load. This then was the major contribution of the cladding to the overall strength of the Garage.

The basic philosophy of not attempting to maximise the cladding stiffness but instead leaving it as normally constructed has still resulted in reducing the DL + SWL case to a less severe case than DL + EWL thereby producing an economical construction. Adding more sheeting fasteners or trough fixing the sheeting instead of crest fixing or adding seam fasteners or some combination of all three of these variations would have stiffened the sheeting but as DL + SWL was not the critical case no advantage would have been gained.

The governing case was DL + EWL and the results for the various load cases follow:

Loads for these cases were in accordance with SAA Code 1170 Part 1, 1971 and Part 2, 1975. Load factors complied with the S.A. Building Act 1976 and these were 1.5 DL + 2.0 LL and 1.2 WL - DL.

DL + LL - Satisfactory for the domestic case which includes the roof maintenance live load and a 1.3 kN concentrated load at any one bottom chord panel point.

- Unsatisfactory for industrial case which increases the concentrated load to 4.5 kN.

DL + EWL - Failure case with maximum load - Adelaide, 50 year return interval, wind category 2, internal pressure 0.4 and load factor 1.2 WL - DL. Failed by weld cracking at the truss to column connection and truss bottom chord buckling.

DL + SWL - Satisfactory as this case proved less critical than DL + EWL due to reduced uplift loads and most of the lateral loads were carried by the sheeting.

0.75 (DL + LL + WL) - Satisfactory as this case was never critical.

The garage as finally tested was without knee and wall bracing and with trusses bolted to the columns.

This particular tested garage then was perfectly satisfactory for the majority of the Adelaide Metropolitan Area and would also suffice for many country areas. (Note: Section 8, Addendum).

Interstate locations may insist on a load factor of 2.0 (WL - DL). The particular tested garage does provide more than this factor for Adelaide, 50 year return interval, thereby 42m/sec., wind category 3 and internal pressure 0.8.

As only one Garage was tested, these conclusions only strictly apply to that tested Garage and caution should be exercised in using the results even for an identical Garage, (i.e. Garage of the same materials, dimensions and style). See Section 6.5 for more detailed explanation.

The structural framework had several faults and the major one, namely the badly positioned knee braces, was overcome by removing them completely. A second major fault causing unnecessary stress was the large eccentricity at the truss-column joint.

This eccentricity could easily be reduced thereby strengthening the frame but increased lateral support would then be required for the bottom chord. The columns could also be strengthened at little cost with more battens placed between the main legs. The addition of correctly positioned knee braces would strengthen the structure.

8. ADDENDUM

The publication of the South Australian Government namely "Regulations under the Building Act 1970 - 1976" was amended in July 1978. The effect of the amendment on this thesis was that the ultimate load combinations of 1.5 DL + 2.0 LL and 1.0 DL - 1.2 WL were replaced by 2.0 (DL + LL) and 2.0 (DL - WL). Therefore this addendum was required to explain the effect of these increased ultimate loads. Relevant parts of the thesis that were affected were Figure 5.1, page 42 and Appendices C and D.

8.1 DL + LL

Appendix C1

2.0(DL + LL) = 12.25 kN plus 2.6 kN concentrated

$$\text{Load Transfer allowed} = \frac{14.25 - (2.5 + 1.75)}{12.25 - (2.5 + 1.75)} = 1.25$$

$$\text{and } \frac{3.4}{2.6} = 1.31$$

Weighted mean value = 1.27

Actual ratio = 1.55

So actual load was only 0.82 of that required. However, the truss did not fail at that load so the truss was either at best not unsafe at all or at worst unsafe by something less than 18%.

Appendix D3

The analysis of Appendix D4 was in accordance with the SAA Code AS 1250 and therefore used the correct ultimate loads and it showed that the structure satisfied the code for the purely Domestic case (no concentrated live load) but required an increased yield stress to satisfy the case of DL + LL (maintenance) + LL (concentrated). That increased yield strength was confirmed by material testing (Appendix A) for the tested garage. It was therefore the author's considered opinion that the particular garage as proof tested and analysed was satisfactory for the full load case of DL + LL (maintenance) + LL (concentrated).

8.2 DL + EWL

Appendix C2

This was for the combination of 2(DL - WL) for Wind Category 3, Internal Pressure 0.8 and so was still applicable.

Appendix D4

This analysed the truss for the actual load that caused the collapse and as that load was higher than 2(DL - WL), Wind Category 3, Internal Pressure 0.8 that analysis was still relevant.

Appendix D5

This analysed the column for the column loading that accompanied the ultimate truss loading. This gave a column load of 7.1 kN whereas the load case of 2.0 (DL - WL), Wind Category 3, Internal Pressure 0.8 gave a column load of 8.2 kN ie a 16% increase. From Appendix D.5.6 it was observed that the top panel of the column was the critical panel and to support the 7.1 kN load needed Fy = 280 MPa. Therefore to support the increased load of 8.2 kN, Fy must be raised from 280 to 320 MPa. The stress of 320 < 360 as used for the truss analysis and confirmed by Appendix A. The column was in fact proof tested under this load of 8.2 kN and found to be satisfactory.

It was therefore the author's considered opinion that the particular garage as proof tested and analysed was satisfactory for 2.0(DL - WL), 0.8 Internal Pressure, Wind Category 3. The garage was however unsatisfactory for Wind Category 2.

8.3 DL + SWL

Appendix C3

This was for the maximum applied load case of Wind Category 2, zero internal pressure and 1.2 WL - DL. The loads for Wind Category 3, 0.8 Internal Pressure and 2.0(WL - DL) were

$$P_1 = 0,\ P_2 = 29.4,\ P_3 = 24.9,\ P_4 = 23.0,\ P_5 = 20.0$$

$$P_1 + P_4 = 23.0$$

It was noted that compared to the worst test load the lateral load was reduced and although the roof loads (P_2, P_3) and the end wall load (P_5) were increased, they were still less than those applied under EWL.

Also worth remembering that the garage was never loaded to failure under DL + SWL.

Appendix D6

This was only concerned with the lateral load and as this was reduced by dropping to Wind Category 3 that analysis is of no further concern. The increased load factor was more than compensated for by the reduction in loads obtained from dropping from Category 2 to Category 3.

It was therefore the author's considered opinion that the particular garage as proof tested and analysed was satisfactory for 2.0(DL - WL), any value of internal pressure of suction, Wind Category 3. The garage was however unsatisfactory for Wind Category 2.

8.4 Summary

Under the increased load factors of 2.0 (DL + LL) and 2.0 (DL - WL) the tested Garage remained structurally sufficient for all load combinations provided that the wind loading was never more severe than that for Adelaide, Category 3. The worst possible combinations of internal and external wind effects were supported by the tested Garage but again only provided that the wind intensity was not greater than Adelaide, Category 3.

APPENDIX A - MATERIAL TESTING

Due to the uncertainty of the yield stress and the modulus of elasticity of the frame component materials, tests were carried out to confirm values recommended by the manufacturers. This took the form of a series of tensile tests on the samples provided with the frame. The samples were cut to length to ensure a length to width ratio of at least ten to one so that yield would occur towards the centre of the sample.

Initial difficulty was encountered in gripping the tube sections without welding on end plates to avoid crushing of the ends. This problem was solved by using the same testing procedure as used by the manufacturers, Tubemakers of Australia Ltd. This required the making of conical plugs which were forced into the ends until the tube took the circular shape of the tapered ends. This ensured that the tubes could be gripped firmly in the jaws of the 600 kN MFL Testing Machine and the load was applied to the circumference of the tube and not just on two opposing faces.

Strain readings were then taken using a 100 mm extensometer of type 'Demec gauge 2285 manufactured by W.H. Mayes and Sons'.

The samples were loaded to yield and then unloaded to test elastic behaviour. Loading then continued to find ultimate tensile strength.

As a result of the tensile tests it was decided to use $E = 2 \times 10^5$MPa for all readings. However stresses above 280 MPa were then corrected using the actual stress strain curve.

Stress Strain curves for 30mm square by 1.6 thick tube and 12mm square solid bar are given in Figures A1 and A2.

The graphs have 2 ranges for strains and the higher range applies to the unloading and reloading cycles.

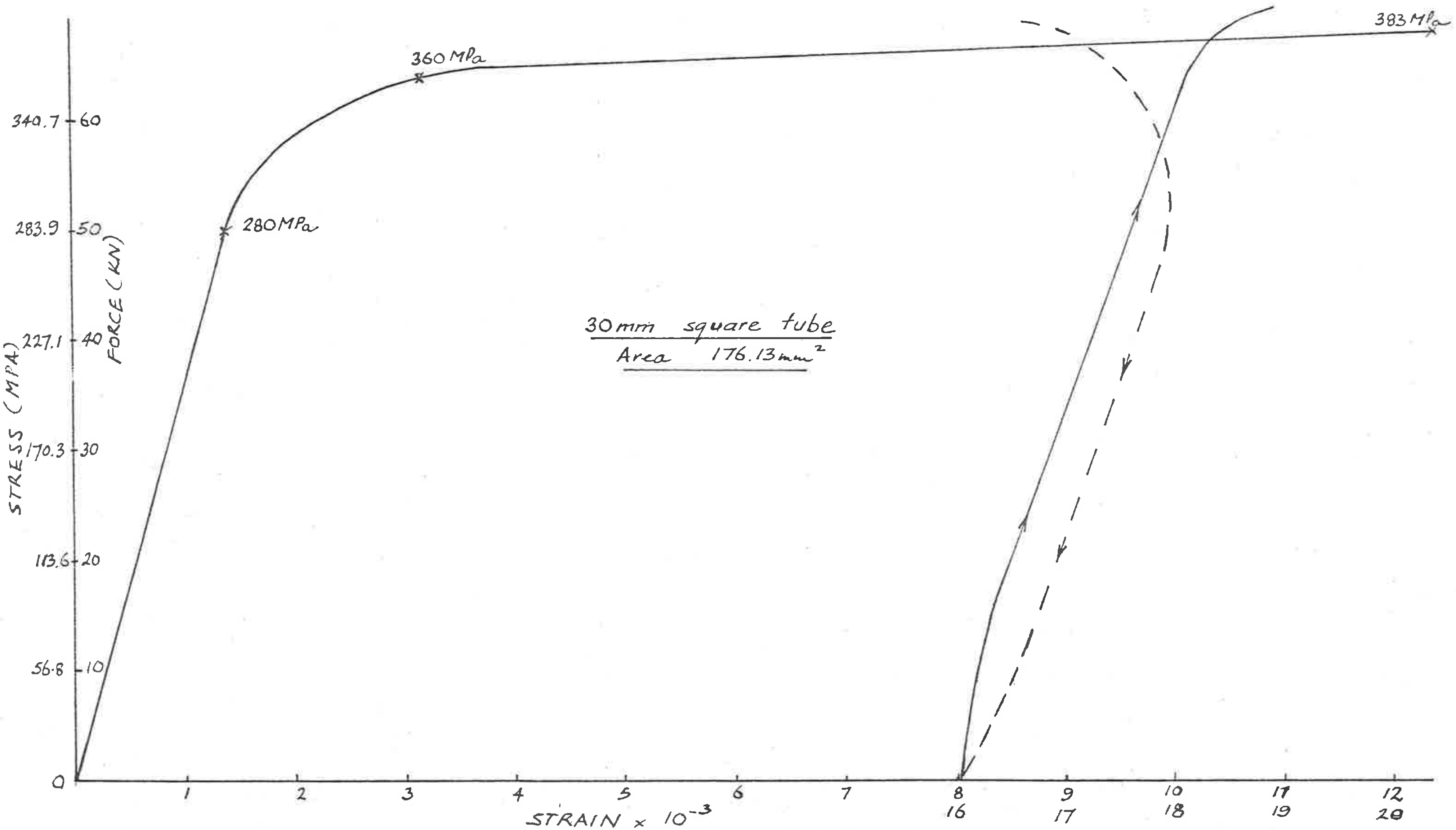

383 MPa
360 MPa
280 MPa
340.7
283.9
227.1
170.3
113.6
56.8
0
60
50
40
30
20
10
STRESS (MPA)
FORCE (KN)
30mm square tube
Area 176.13 mm²
1
2
3
4
5
6
7
8
9
10
11
12
16
17
18
19
20
STRAIN × 10^{-3}

FIGURE A1

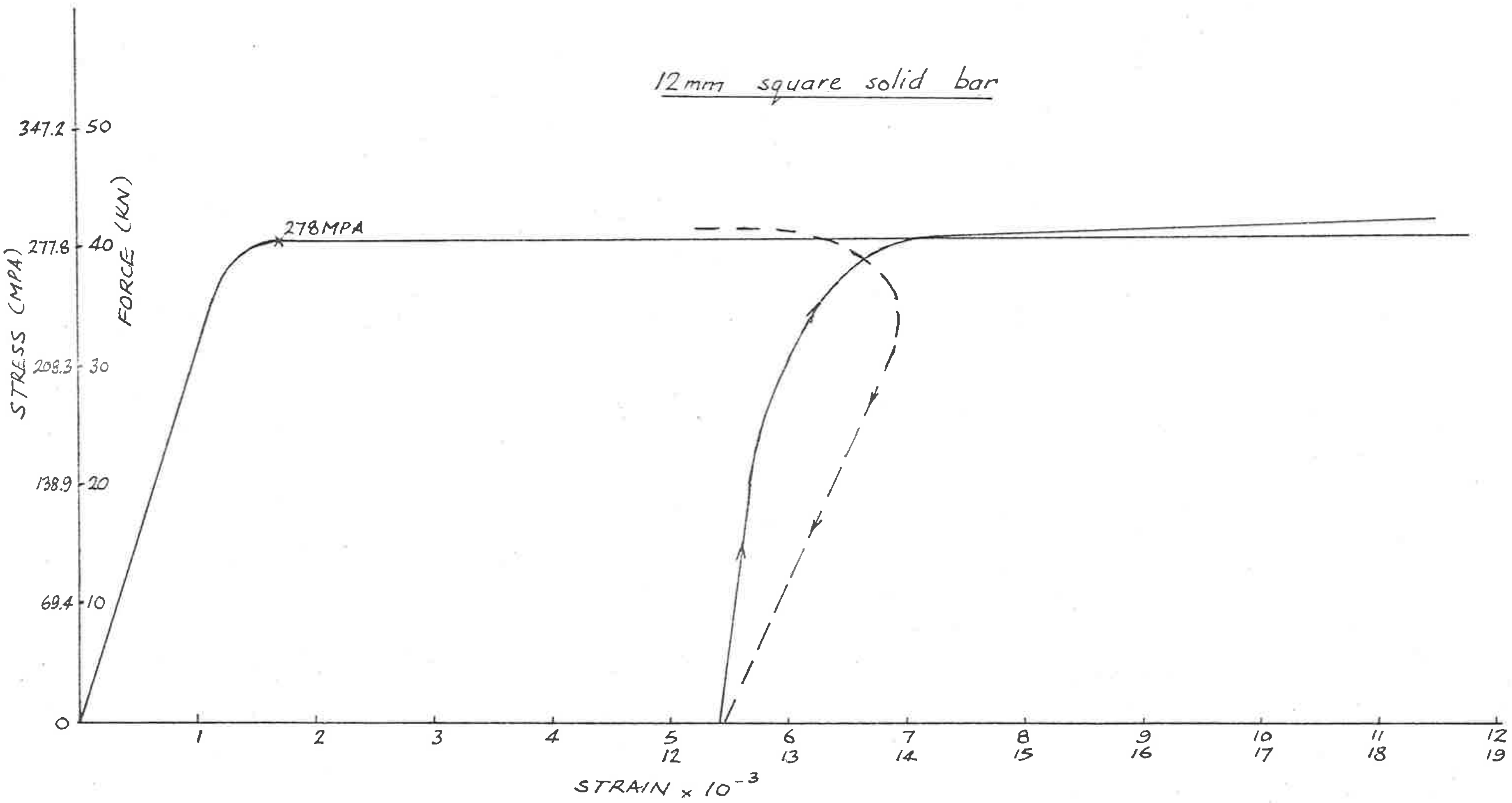

12mm square solid bar
347.2
50
277.8
40
208.3
30
138.9
20
69.4
10
0
STRESS (MPA)
FORCE (KN)
278MPA
1
2
3
4
5
12
6
13
7
14
8
15
9
16
10
17
11
18
12
19
STRAIN × 10^-3

FIGURE A2

APPENDIX B - COMPUTER PROGRAM

B1 General

A computer program was developed to process the many readings recorded in these series of tests. By recording the test results on computer paper tape with the Facit tape puncher, it was possible to transfer the results to disc file on the S.A.I.T. Cyber computer, using the PDP8/E mini-computer. The data was then edited with the Cyber computer and each set of load readings stored on a separate disc file. This made it possible to call-up each file as it was required for processing.

The program was then set up to read the required information from the applicable disc file. The program had the capability of re-arranging the data into its correct sequence of positions, as required for Section 3, but not required for Section 5, as the faulty potentiometers had been corrected and so all gauges could be put directly into the correct order.

With the strain gauges in correct position pairs, the readings were then made compatible by linearising them and then the average zero reading was subtracted from each reading to produce the absolute strain value.

Once this stage was reached, all readings were compatible, positioned correctly and the readings in absolute micro-strain. This then completed the work of the primary program and then sub-routines AXIAL F and MOMENT were called up to determine the axial force and moment for each pair of gauges.

B2 Basis of Interpreting Strain Readings

The stresses were obtained from the strains using basic principles;

Stress = Strain x Modulus of Elasticity

Using $E = 2.10^5$ MPa

Once the stresses were found the axial forces and bending moments in the plane of the frame could be calculated using:-

(a) Axial force stress = average of the sum of the stresses of the top and bottom gauges

(b) Bending Stress = average of the difference of the stresses between the top and bottom gauges

Then from simple bending theory:-

(a) Axial force = axial stress x area

(b) Moment = bending stress x section modulus

These operations were performed by the program.

B3 Sign Convention

A convenient convention was used to indicate the direction of forces and bending.

Axial Force:- positive strain indicates tension in the member

Moments:- positive value for moment indicates tension on the top gauge

Final results from the program were presented in order of load increments and positions on the frame.

To assist in the analysis of the results, the total applied load was also produced by the program.

The actual program wasnot presented here as it wastoo wide for this sheet width and retyping it caused confusion. However the program is available for perusal.

APPENDIX C - LOAD TEST RESULTS

C1. DL + LL

The stresses, end panel moments and forces for the largest recorded loading are shown in Figure C.1.

Except where noted as at the centreline all values are at the strain gauge locations.

NOTE:

1. + Stresses as > 280 MPa have been adjusted for non-linearity.
2. Stresses lower than actual as the gauges were zeroed under 1.0 DL. End shear is 5.00 kN and would be increased to 6.04 kN if 1.0 DL added. i.e. increased by 21 per cent.
3. Loads P1 and P2 are applied to truss T2 only and some of this load is transferred to other trusses via the purlins.

   1.5 DL + 2.0 LL (Roof Maintenance) = 3.75 + 7.25

   $= 11.0$ kN

   2.0 LL (Concentrated) = 2.6 kN

   Applied $= 2.50 + 1.75 + \frac{5}{4} \times 8.0 = 14.25$ kN (See Figure C1)

   plus 3.4 kN concentrated

   $$\text{Load Transfer Allowed} = \frac{14.25 - (2.5 + 1.75)}{11.0 - (2.5 + 1.75)} = 1.48$$

   $$\text{and } \frac{3.4}{2.6} = 1.31$$

   Weighted Mean Value = 1.44

   $$\text{Actual Ratio} = \frac{(\frac{8}{2} + 3.4 \times 0.8)}{5.15 - \frac{1.75}{2}} = 1.57$$

   $$\text{Required end shear} = \tfrac{1}{2}\,(7.25 + 3.75)\ {}^{5}/_{6} + 2.6 \times \frac{4}{5}$$

   $= 6.66$ kN

   Actual end shear $= 5.00 + 1.25 \times {}^{5}/_{6} = 6.04$ kN.

So the actual load was only 0.91 of the required load. However, the truss did not fail under that load so the truss could reasonably be assumed as satisfactory.

4. Horizontal reactions at the truss to column joint did not close and the reason why follows: the vertical reactions at the truss to column joint did not agree because the wall sheeting transferred load from the loaded column to the other columns via the common end purlin and top girt. If the column load is increased to 5.1 kN then the column end moment =

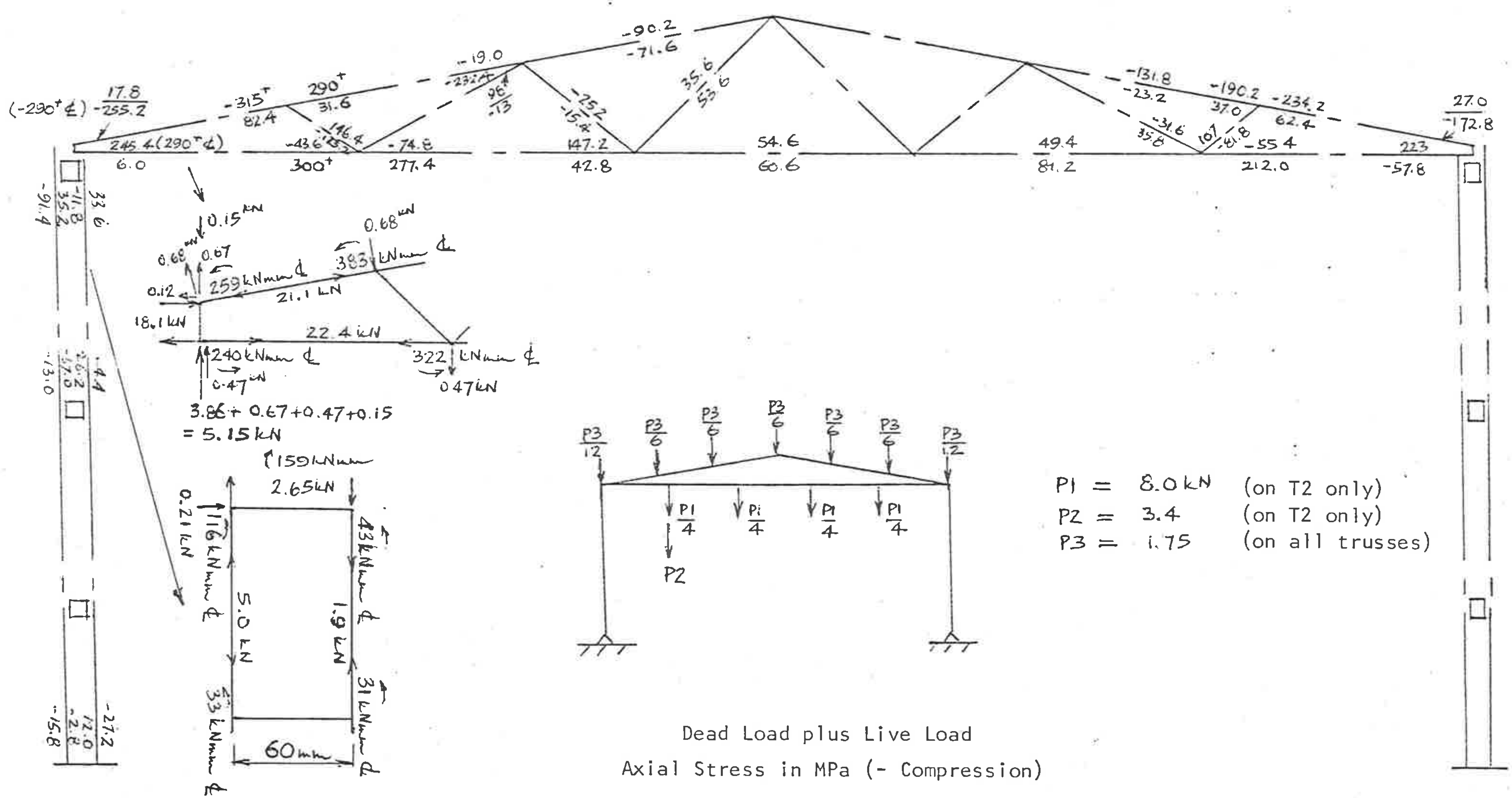
$$\frac{5.1}{3.1} \times 159 = 261 \text{ kNmm}$$

Therefore Truss horizontal force = $\frac{261}{42}$ = 6.2 kN

Horizontal reaction then = 11.9 + 6.2

= 18.1 < 20.9 kN

Gauges 1L were already considered doubtful as they were too close to the member end and if the moment there was higher then agreement could be obtained.



Dead Load plus Live Load

Axial Stress in MPa (- Compression)

Figure C1

C2 DL + EWL

The stresses at the strain gauge locations are shown on Figure C2.

Load case was Wind Category 3, internal pressure 0.8 and load factor 2.0.

Failure load of Wind Category 2, internal pressure 0.4 and load factor 1.2 gave P1 = 21.3, P2 = 31.5.

NOTE:

1. + stresses as > 280 MPa have been adjusted for non-linearity
2. Stresses higher than actual as gauges were zeroed under 1.0 DL.

   1.0 DL = 2.5 kN/Truss

   $$\text{Roof Applied WL} = \frac{29.4 \times 2}{3} = 19.6 \text{ kN/Truss}$$

   $$\text{so} \quad \frac{\text{actual load}}{\text{strain gauge load}} = \frac{17.1}{19.6} = 0.87$$

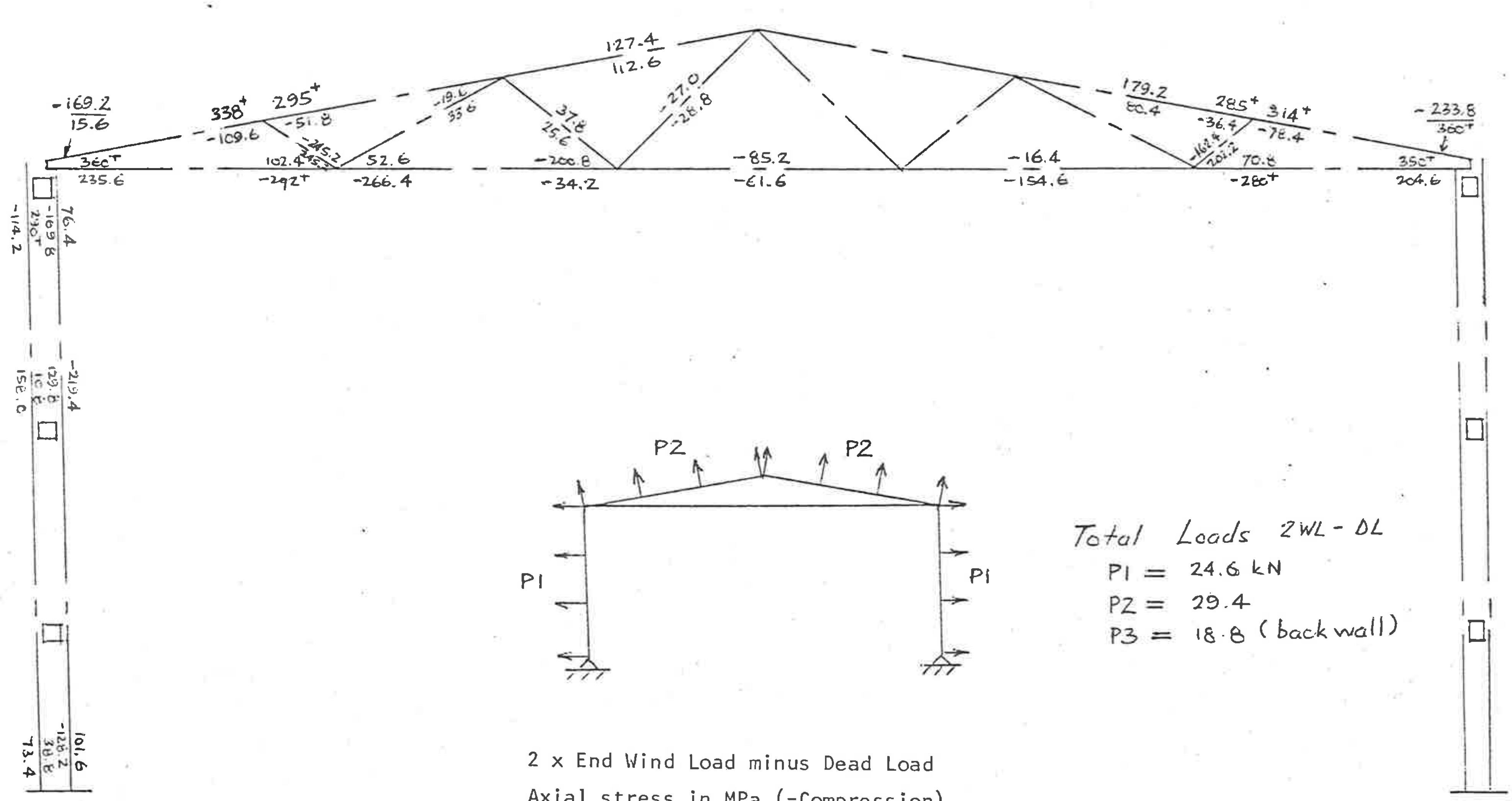
   Truss failed at WL = 31.5 kN

   $$31.5 \times \frac{2}{3} - 2.5 = 18.5 \text{ kN}$$

   $$\text{Now} \quad \frac{18.5}{19.6} = 0.94$$

   $$\text{So} \quad \frac{\text{actual load at failure}}{\text{load giving stresses in Fig. C2}} = 0.94$$

3. Stresses at centrelines not given due to partly plastic, partly elastic behaviour between stresses at strain gauges and stresses at centrelines.
4. Stresses at 1L not reliable as truss was strengthened here after weld failure under DL + LL and also from other results this gauge was located too close to the member end.
5. Stresses not given at 2R due to faulty gauge.



2 x End Wind Load minus Dead Load
Axial stress in MPa (-Compression)

Figure C2

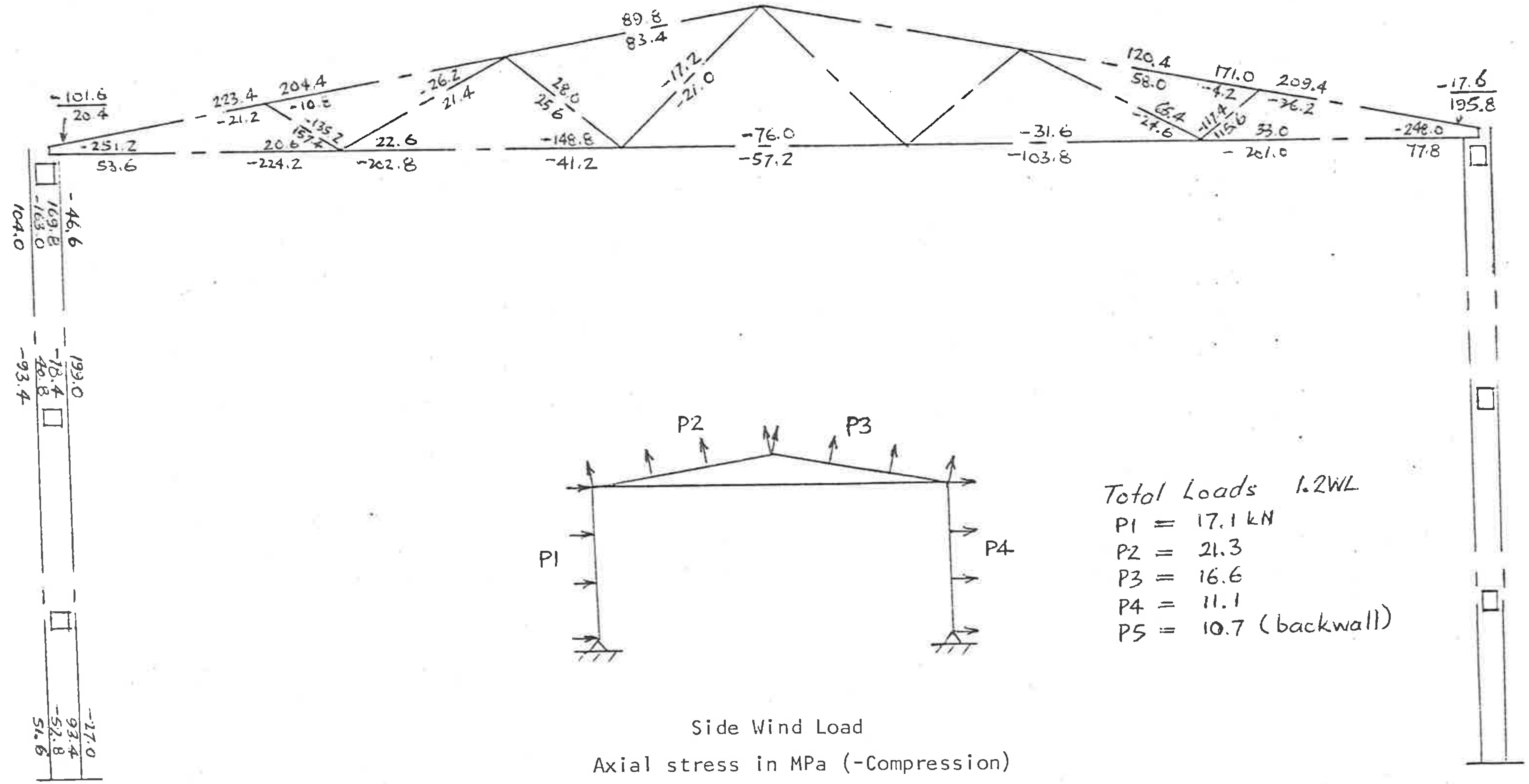
C3 DL + SWL

The stresses for the strain gauge locations are shown in Figure C3a for pinned bases and C3b for fixed bases.

Load case for both diagrams was Wind Category 2, internal pressure 0 and load factor of 1.2. However note that for the fixed base the loads were 9 per cent low - due to insufficient allowance for friction through the loading apparatus.

NOTE:

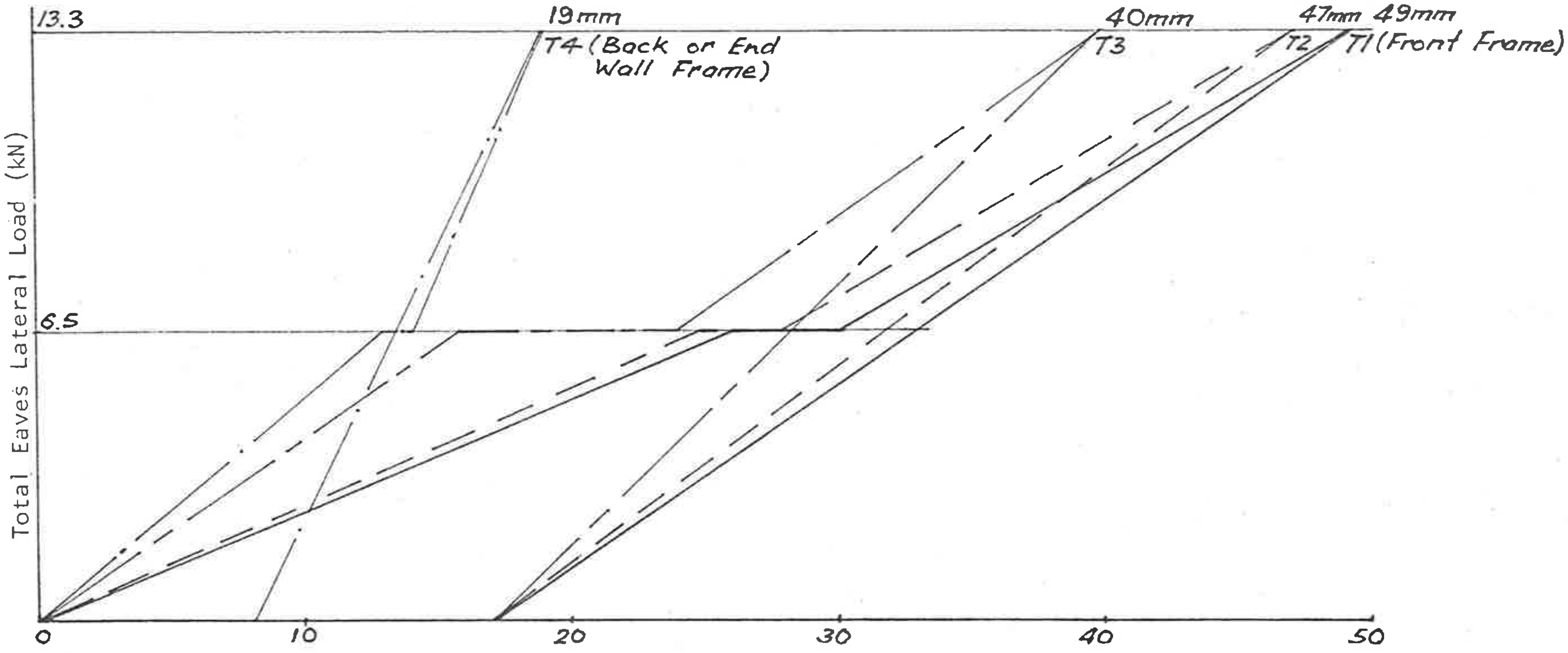
1. No stresses are > 280 MPa so no adjustment was made for non linearity.
2. Stresses higher than actual as gauges were zeroed under 1.0 DL.
3. Equilibrium check at beam column joint was not possible as there were only sufficient gauges at the left hand end for this check and the gauges on 1L were not reliable (see C2 note 4).
4. Column stresses for both bases were almost identical indicating that the fixed base was not effective.
5. Stresses not given at 2R due to a faulty gauge.



Side Wind Load

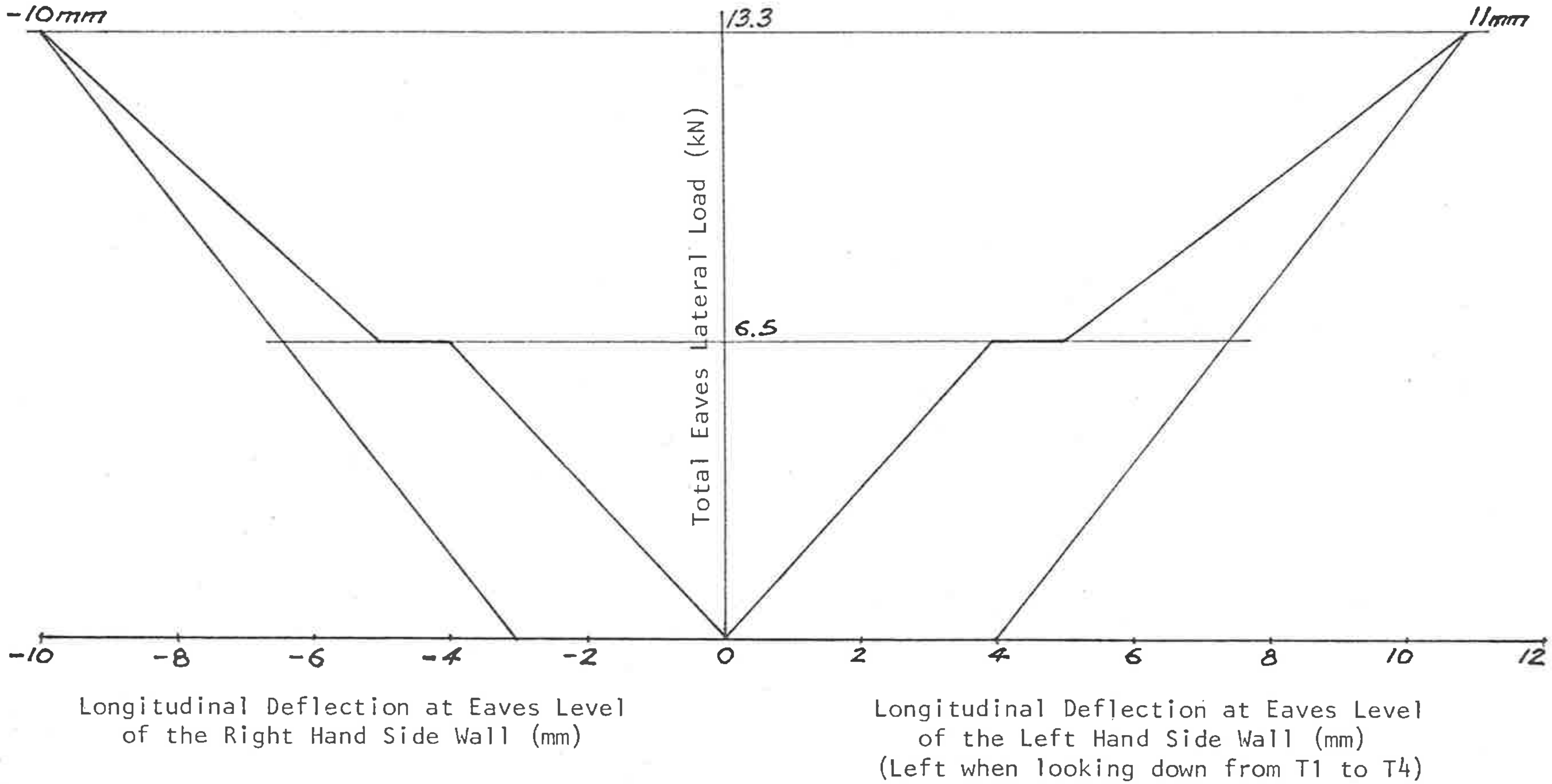
Axial stress in MPa (-Compression)

Figure C3a



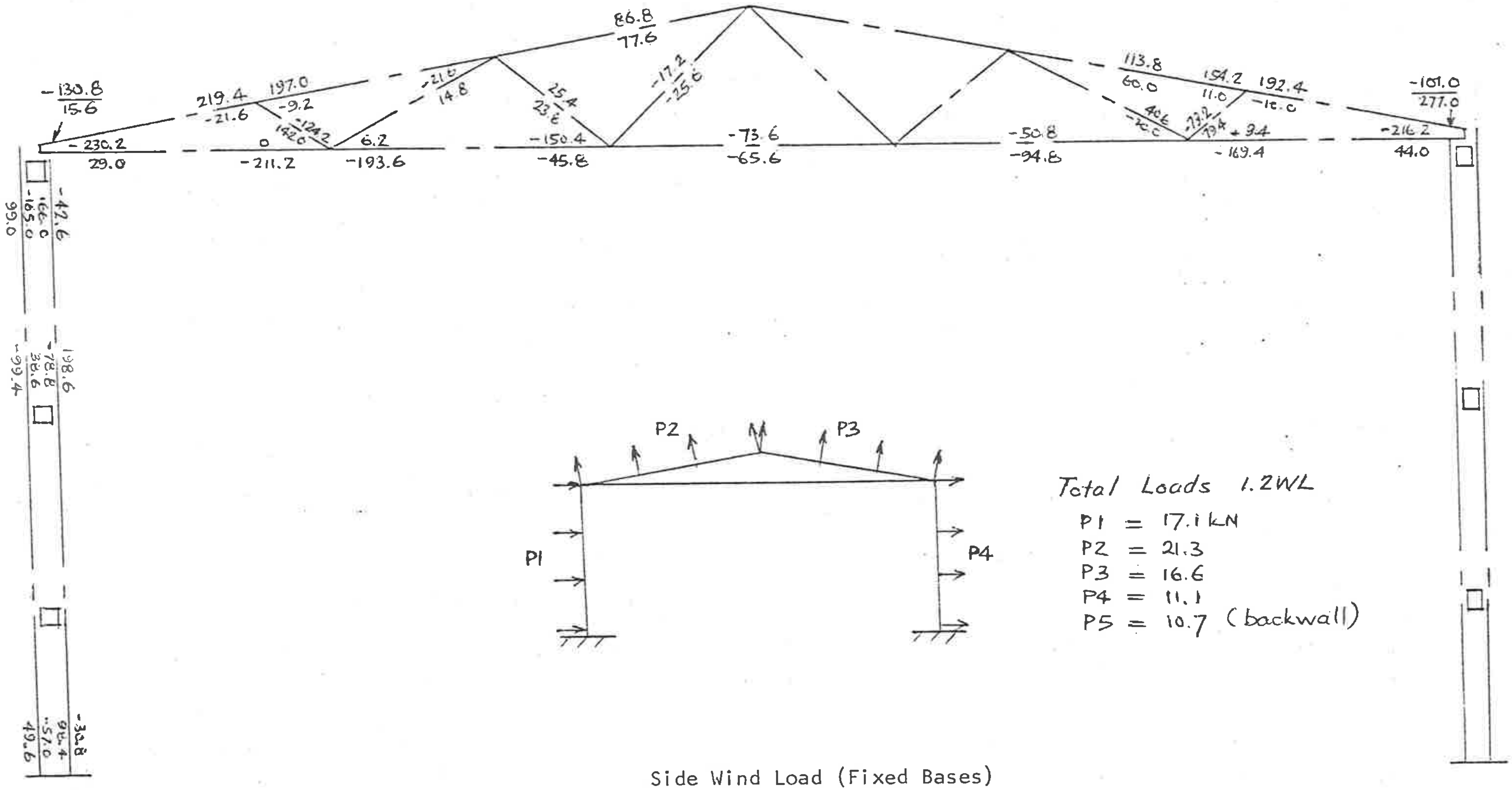
Lateral Deflections (mm) at Eaves Level
Side Wind Load (Pinned Bases)

Figure C3b



Side Wind Load (Pinned Bases)

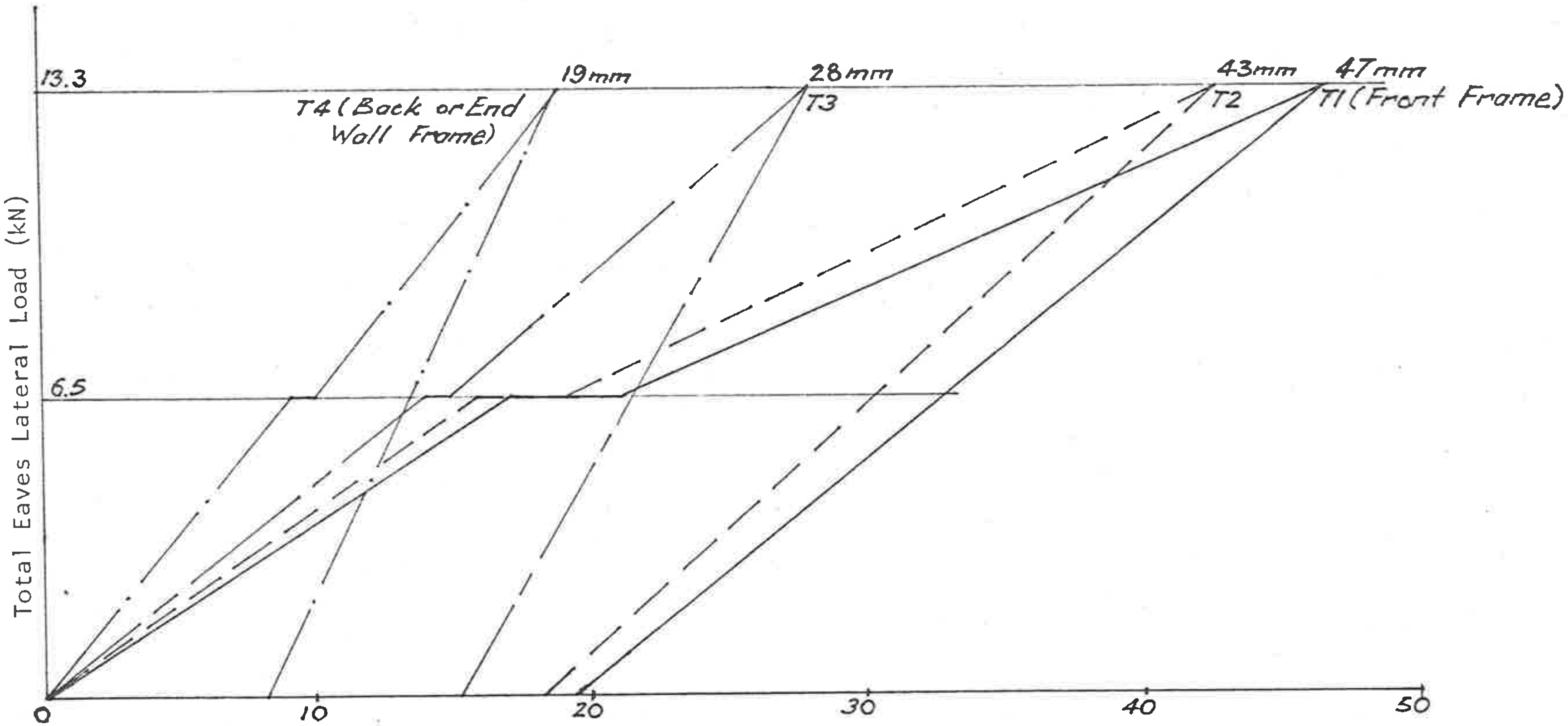
Figure C3c



Side Wind Load (Fixed Bases)

Axial stress in MPa (-Compression)

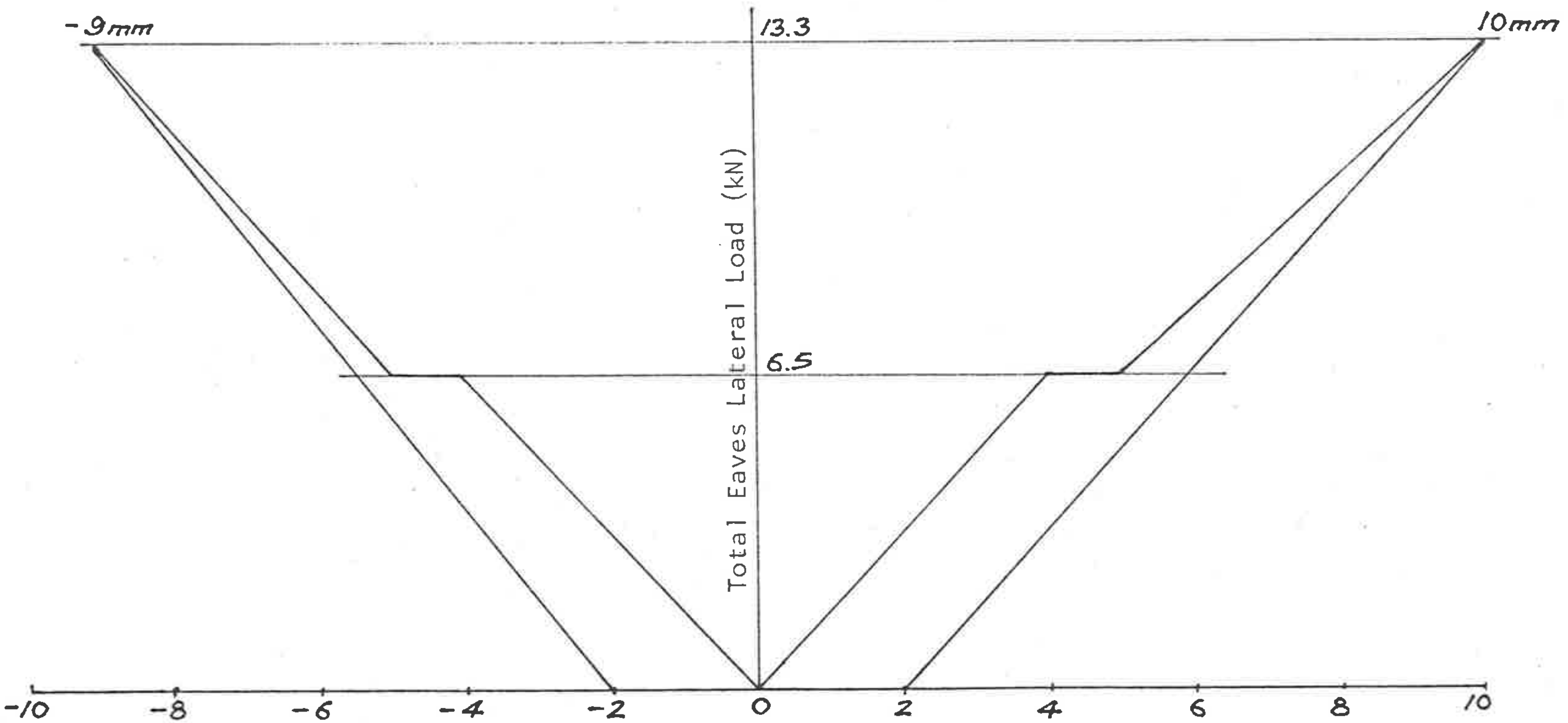
Figure C3d



Lateral Deflections (mm) at Eaves Level

Side Wind Load (Fixed Bases)

Figure C3e



Longitudinal Deflection at Eaves Level of the Right Hand Side Wall (mm)

Longitudinal Deflection at Eaves Level of the Left Hand Side Wall (mm) (Left when looking down from T1 to T4)

Side Wind Loading (Fixed Bases)

Figure C3f

APPENDIX D - STRUCTURAL REVIEW

Member sizes and frame arrangement were determined by the Manufacturer and so the following is a review of the structure as tested.

Frame Arrangement (see Figure D1.)

Abbreviations used in this review

DL - dead load

LL - live load

WL - wind load

EWL - end wind load

SWL - side wind load

D1 Purlins

Material	Timber-Kapur (Hardwood)	
Span	2.44m, spacing 0.98m	
DL	Sheeting (Custom Orb)	.054
	Purlin (60 x 40mm)	.023
		.077 kN/m

LL $\left(\frac{1.8}{0.98 \times 2.44} + 0.12\right)$ = .855 (Reference 16, Part 1 Clause 3.8.1.1.)

DL + LL = .932 kNm

WL Review for the maximum test load of location Adelaide, 50 year return interval, Category 2 (Reference 16, Part 2)

$$C_p = 1.3$$

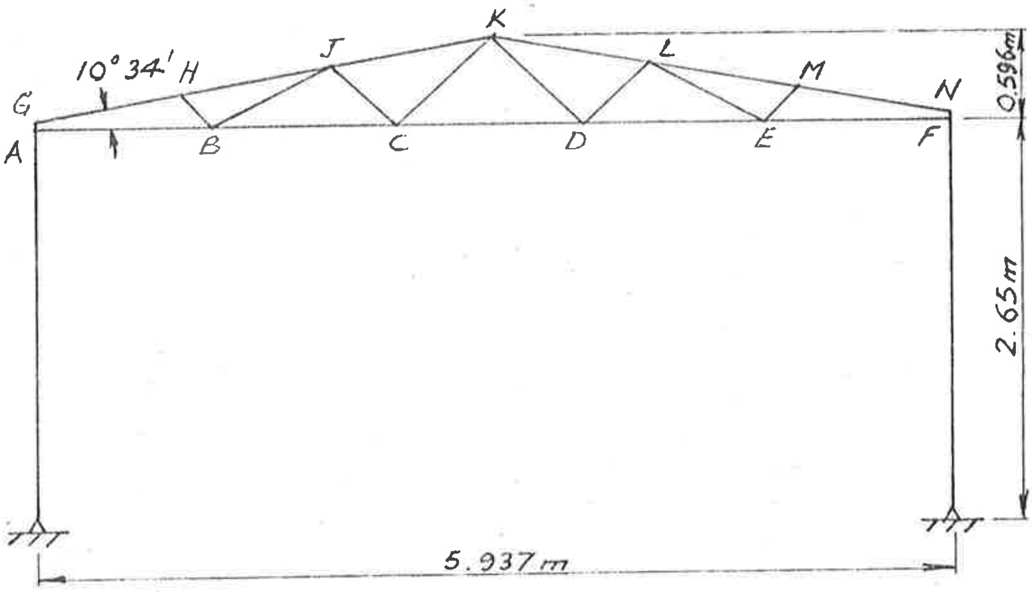
$$V_z = 0.93 \times 42 = 39.1 \text{ m/sec}$$

$$q_z = 0.6 \times \frac{39.1^2}{10^3} = 0.915 \text{ kPa}$$

WL = 1.3 x 0.915 x 0.98 = 1.17 kN/m

DL - WL = -1.09 kN/m

Purlins were continuous over 3 spans



All lines are centrelines
3 bays at 2.44m centres
7.32m overall

FIGURE D 1

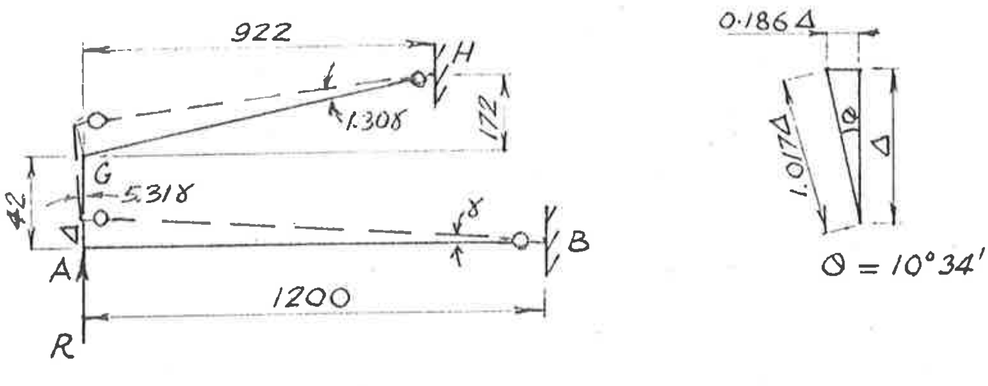


FIGURE D 2

For maximum moment place the live load on the first two spans only but must place the dead load and the wind load on all three spans simultaneously.

Considering DL + LL

BM_{max} = $0.1 \times .077 \times 2.44^2 + 0.117 \times 0.855 \times 2.44^2$ (Reference 23, Page 57)

= $0.045 + 0.595$

= 0.641 kNm (at first internal support)

$Z = 40 \times \frac{60^2}{6} = 24000 \text{ mm}^3$

$f_b = \frac{641 \times 10^3}{24000} = 26.7 \text{ MPa}$

1.5 load duration factor (6 hours duration) (Reference 13, Table 2.4.1.1.)

Stress Grade = $\frac{26.7}{1.5}$ = 17.8 MPa

Green Selected 17 MPa allowable (Reference 13, Table B1)

Seasoned Standard 17 MPa allowable

This DL + LL case is very severe as it assumes that the live load is applied for six hours and even so the overstress of 4.7% is small and therefore the purlins can be regarded as satisfactory for DL + LL.

Considering DL - WL

$BM = 0.1 \times 1.09 \times 2.44^2 = 0.649 \text{ kNm}$

$f_b = \frac{649 \times 10^3}{24000} = 27.0 \text{ MPa}$

2.0 load duration factor (5 seconds duration)

Stress Grade = $13.5 < 17.8$

So DL + LL governs

Conclusion for Purlins

Section 40 x 60mm Kapur Timber of F17 Stress Grade is satisfactory for DL + LL and also for DL - WL for 50 year return interval, location Adelaide, Wind Category 2 and 0.4 Internal Pressure.

D2 Girts

Span 2.44m, Spacing 1.0m

The highest test load was with

C_p = 1.0 and Wind Category 2

Girts continuous over 3 spans

$$WL = 1.0 \times 0.915 \times 1 = 0.915 \text{ kN/m}$$

$$BM = 0.1 \times 0.915 \times 2.44^2$$

$$= 0.545 \text{ kNm}$$

$$Z = 60 \times \frac{40^2}{6} = 16 \times 10^3$$

$$f_b = \frac{545}{16} = 34.05 \text{ MPa}$$

2.0 load duration factor

Stress Grade = 17.0

Conclusion for Girts

Section 60 x 40mm Kapur Timber of F17 Stress Grade is satisfactory for 50 year return interval, location Adelaide, Wind Category 2, with C_p of 1.0.

D3 Truss, DL + LL

D3.1 Loads and Design Method

DL	sheeting	0.131
	purlins	0.057
	truss	0.230
		0.418 say 0.42 kN/m

LL 0.25 x 2.44 = 0.61 kN/m plus 1.3 kN concentrated at any bottom chord panel point taken one at a time. (Reference 16, Part 1, Clauses 3.8.1.1. and 3.8.3.2.).

Due to eccentricity at the truss column joint the end panel of the truss was able to deflect vertically as a rigid frame. Therefore the two end panels must be analysed as sway frames and the internal panels as forming a rigid jointed truss.

Sway Equation for End Panels (see Figure D2).

$$(M_{AB} + M_{BA}) \frac{\Delta}{1200} + (M_{AG} + M_{GA})\left(\frac{-0.186\Delta}{42}\right) + (M_{GH} + M_{HG}) \frac{1.017\Delta}{938}$$

$$+ R.\Delta = 0$$

$$(M_{AB} + M_{BA}) - 5.31\ (M_{AG} + M_{GA}) + 1.30\ (M_{GH} + M_{HG}) + 1200R = 0$$

Individual moments

$$M_{AB} = M_{BA} = -\ 6EI_1 \frac{\Delta}{\ell_1{}^2} \qquad \ell_1 = 1200$$

$$M_{GH} = M_{HG} = -\ 6EI_1 \frac{1.017\Delta}{\ell_2{}^2} \qquad \ell_2 = 938$$

$$I_1 = 24000$$

$$\frac{M_{GH}}{M_{AB}} = 6EI_1 \frac{1.017\Delta}{938^2} \times \frac{1200^2}{6EI_1\Delta} = 1.664$$

$$M_{AG} = 6EI_2 \frac{0.186\Delta}{\ell_3{}^2} \qquad \ell_3 = 42, \quad I_2 = 13453$$

$$\frac{M_{AG}}{M_{BA}} = 6E \times \frac{13453 \times 0.186\Delta}{42^2} \times \frac{1200^2}{6E \times 24000 \times \Delta} = 85.1$$

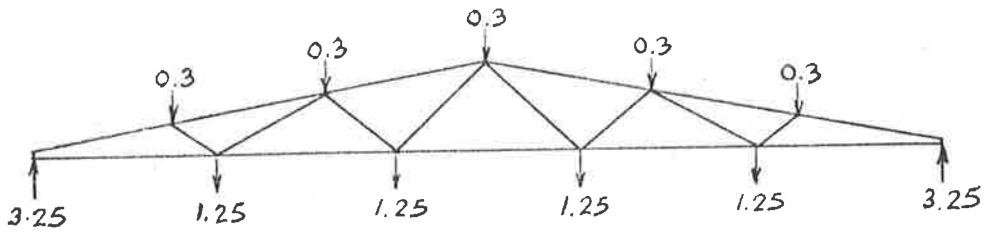
$$M_{AB} : M_{AG} : M_{GH} = 1 : -\ 85.1 : 1.664$$

ACES computer package (Reference 12) was used for all analysis from from here on. An initial manual analysis was done as a check against ACES and good agreement was obtained.

D3.2 Comparing Analysis Results against Test Results

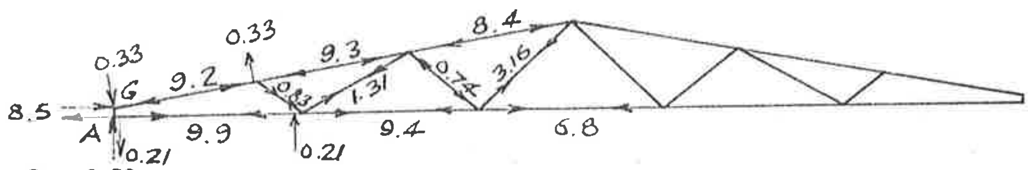
Member	GH	HG	AB	BA
Axial load				
Analysis	10.7	10.7	10.4	10.4
Test	10.7	10.7	11.5	11.5
Ratio	1.0	1.0	0.91	0.91
Moment				
Analysis	-248	130	-189	103
Test	-227	129	-190	103
Ratio	1.09	1.0	1.0	1.0

For full results see Figures D3, D4



LOADS (kN)

Note: Bottom Chord loads were applied to one truss only, namely T2, and as this truss deflected part of its bottom chord loading was transferred via the purlins to the adjacent trusses. Nett loads were determined below, i.e. 2.22 not 3.25 at the support.



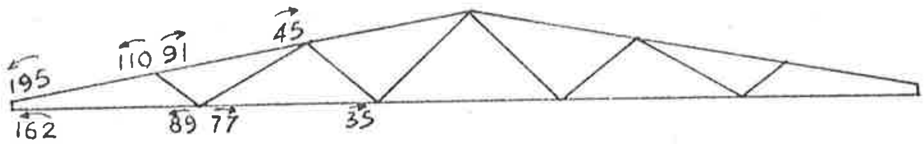
$\Sigma V = 0 \rightarrow 2.22$

$H_G = \overleftarrow{0.48}$, $H_A = \overrightarrow{1.4}$

i.e. $\Sigma H \neq 0$

SHEAR FORCE AND AXIAL FORCE (kN)

(Calculated from Strain Gauge Readings)

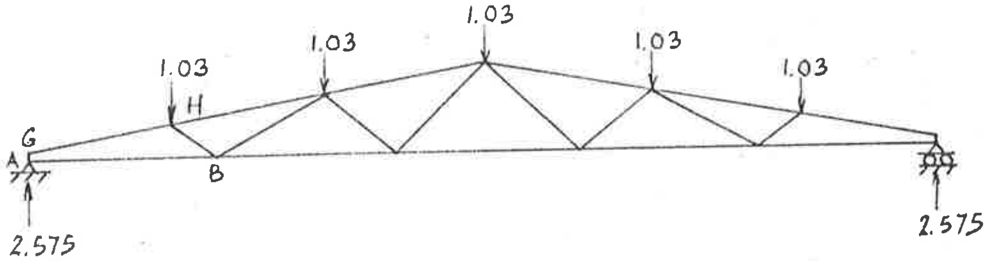


MOMENTS (kNmm)

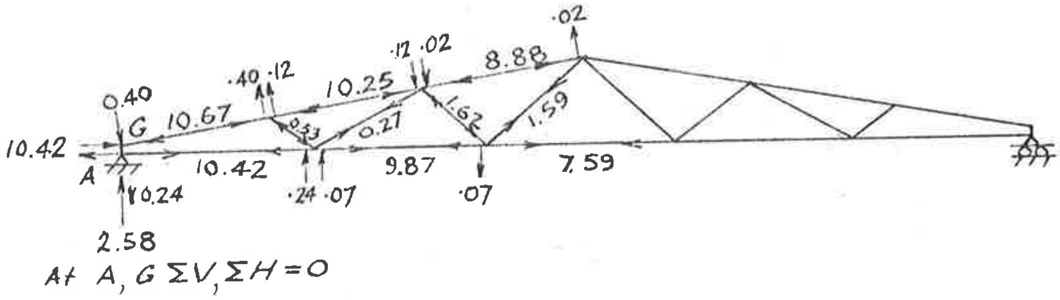
(Calculated from Strain Gauge Readings)

DL + LL - Load Test, Pinned Bases, No Knee-Braces, Assembled Garage

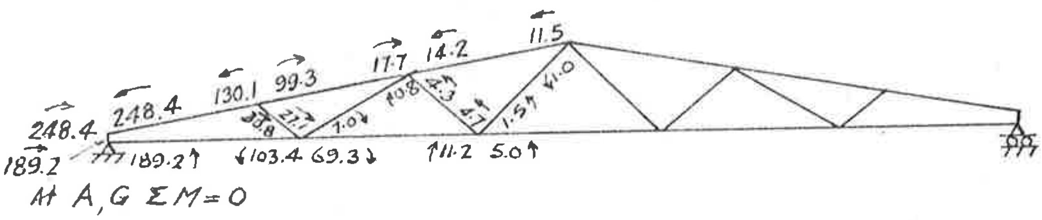
FIGURE D3



LOADS (kN)
Loads comprise Dead Load plus Roof Live Load



SHEAR FORCE AND AXIAL FORCE (kN)



MOMENT (kNmm)

DL + LL - Analysis by ACES, Bare Truss, No Columns

FIGURE D4

Note: (1) Comparison was only on the end panels as the loading arrangements for test and analysis differed except at the end panels.

(2) Test load/Analysis load $= 0.86 = (\frac{2.22}{2.575})$

Table figures on page 47 are adjusted so that they are for the same load.

(3) Agreement was excellent except for the axial force in AB and the moment at GH.

(4) From test result $\sum H \neq 0$ and on reflection gauge GH was only 60mm from the column centreline whereas gauge AB was 105mm from the column centreline. Gauge GH was probably too close to give a true reading. Increasing GH moment by 10% gives $\sum H = 0$ at G. This leaves $\sum H \neq 0$ at A but this error was less than as shown on Figure 6.3 as the 8.5 kN horizontal reaction is boosted when moment at GH is increased and the balance possibly results from an external horizontal reaction at the truss to column connection.

D3.3 Further Comparison of Test Results against Analysis Results

Consider the 1.3 kN Point Load applied at the second panel point.

A load test was done for this case (except load was 1.81 kN not 1.3 kN) on the bare truss i.e. without columns or sheeting. Supports were specially fabricated to give the true simple supports assumed in the analysis. Therefore after multiplying the test results by $^{1.3}/_{1.81} = 0.72$ they are compared with the analysis results in the following table.

Member	GH	HG	AB	BA
Axial Load				
Analysis	3.3	3.3	3.2	3.2
Test	2.9	2.9	3.0	3.0
Ratio	1.14	1.14	1.07	1.07
Moment				
Analysis	77	39	58	31
Test	68	34	61	36
Ratio	1.13	1.15	0.95	0.86

Note: (1) Agreement generally satisfactory

(2) Previous comparison (Figures (D3 and D4) suggested GH was reading low so this correction would improve the comparison

(3) Equilibrium at A, G was not fully satisfied from the test results but was close enough not to produce any significant errors.

(4) Alignment errors of the actual truss (see Figure 3.1) were not allowed for in the Analysis Results as their effect was considered to be negligible.

(5) For full results see Figures D.5, D.6, D.7.

D3.4 Member Investigation

Using Analysis Forces and Moments

Truss

Chords - GH Critical - Concentrated load placed at the 1st Panel Point

$P = 10.77 + 4.33 = 15.1$ kN (Sum of figures D4 and D7)

$M = 253 + 101 = 354$ kNmm

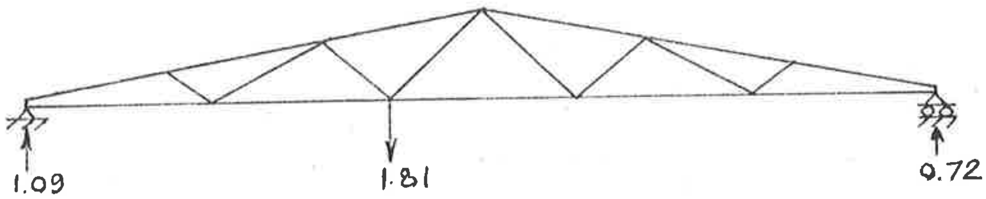
Section Area $= (30^2 - 26.88^2) = 177.5\ mm^2$

$I = \frac{1}{12}(30^4 - 26.88^4) = 24000\ mm^4$

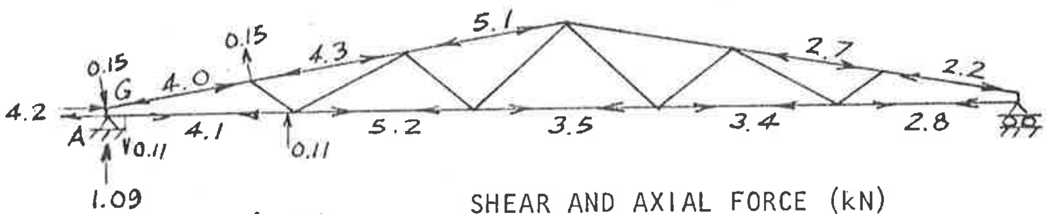
$Z = 1600\ mm^3$

$r = 11.63$ mm

$G_A = 0,\quad G_b = 0.86$ (Reference 15, Appendix E)



LOADS (kN)



SHEAR AND AXIAL FORCE (kN)

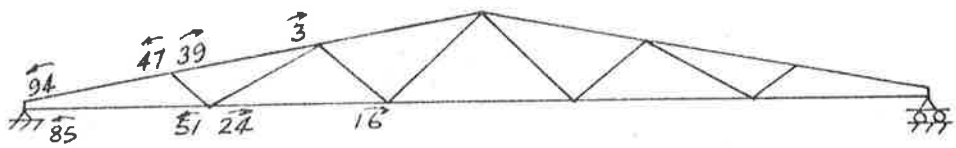
(Calculated from Strain Gauge readings)

At G $\Sigma V = 0.10 \neq 0$

$\Sigma H = 0.30 \neq 0$

At A $\Sigma V = 0.1 \neq 0$

$\Sigma H = 0.1 \neq 0$

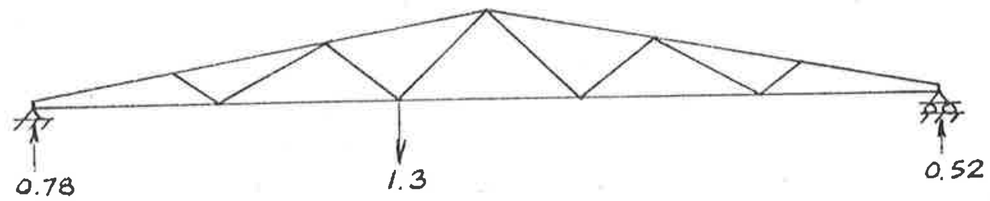


MOMENTS (kNmm)

(Calculated from Strain Gauge readings)

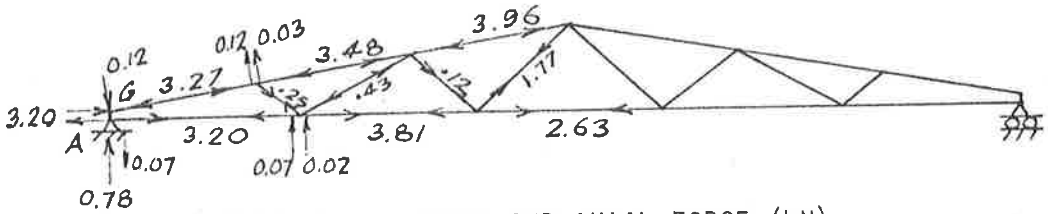
DL + LL - Concentrated LL, Load Test, Bare Truss, No Columns

FIGURE D5



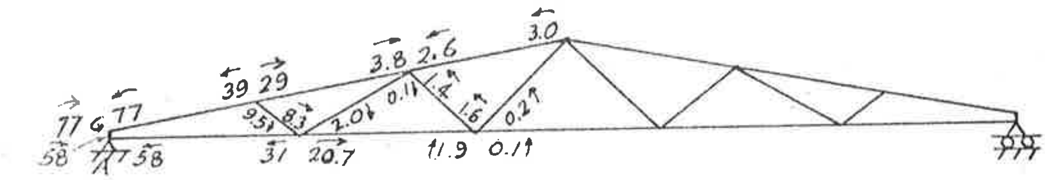
LOADS (kN)

Load is the Bottom Chord Live Load



At A, G $\Sigma V, \Sigma H = 0$

SHEAR AND AXIAL FORCE (kN)

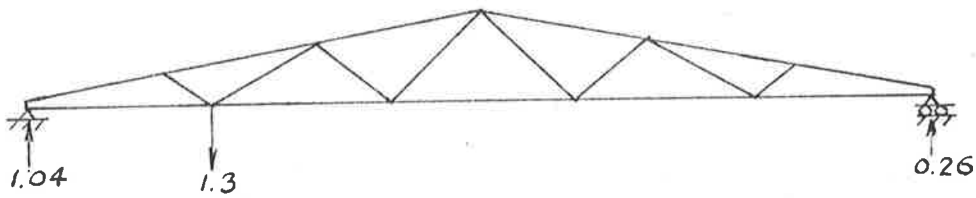


At A, G $\Sigma M = 0$

MOMENTS (kNmm)

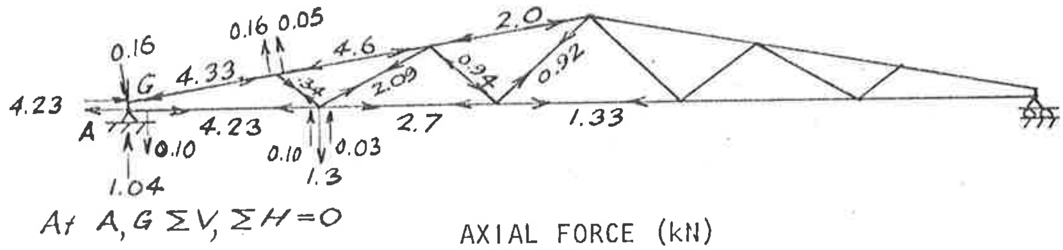
DL + LL - Concentrated LL, Analysis by ACES, Bare Truss, No Columns

FIGURE D6

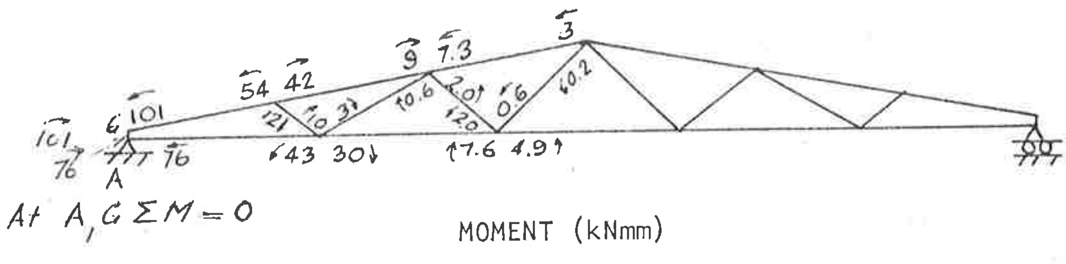


LOADS (kN)

Load is the Bottom Chord Live Load



AXIAL FORCE (kN)



MOMENT (kNmm)

DL + LL - Concentrated LL - Analysis by ACES, Bare Truss, No Columns

FIGURE D7

Sway not prevented in the vertical plane

so $\ell = 1.13L$

$$\ell/r = 1.13 \times \frac{940}{11.63} = 91$$

$$F_{oc} = \frac{\pi^2 E}{(\ell/r)^2} = \frac{\pi^2 \times 2.10^5}{91^2} = 238 \text{ MPa}$$

$$F_{ac} = \frac{QF_y}{\Omega}\left(\left(\frac{1.25 + \frac{F_{oc}}{QF_y}}{2}\right) - \sqrt{\frac{\left(1.25 + \frac{F_{oc}}{QF_y}\right)^2}{4} - \frac{F_{oc}}{QF_y}}\right)$$ (Reference 14, Clause 3.6.1.)

$Q = 1$, $F_y = 250$

$$F_{ac} = 0.6 \times 250 \left(\frac{\left(1.25 + \frac{238}{250}\right)}{2} - \sqrt{\frac{\left(1.25 + \frac{238}{250}\right)^2}{4} - \frac{238}{250}}\right)$$

$$= 89 \text{ MPa}$$

Considering the section elastically

$$f_{ac} = {}^{15.1}/_{.1775} = 85 \text{ MPa} < 89 \text{ MPa}$$

Bending $f_b = \frac{354 \times 10^3}{1600} = 221$ MPa (Reference 14, Clause 3.1.1.)

$$F_b = 0.6 \times 250 = 150 < 221$$

At the support

$$\frac{f_{ac}}{0.6F_y} + \frac{fb_x}{Fb_x} = \frac{85}{150} + \frac{221}{150}$$ (Reference 14, Equation 3.7.1(c))

$$= 2.04 > 1.0$$

Section is unsatisfactory for both moment alone and moment plus force.

Now consider the ultimate load case and remove initially the 1.3 kN concentrated load

$$P_u = \frac{10.77}{0.6} = 17.9 \text{ kN}$$

$$M_u = \frac{253}{.6} = 422 \text{ kNmm}$$

Centroid of ½ area from XX axis

$$88.75y = 46.8 \times 14.22 + 41.95 \times 6.72$$

$$y = 10.67\text{mm}$$

$$Z_p = \frac{A}{2} \times y \times 2$$

$$= 88.75 \times 10.67 \times 2 = 1895 \text{ mm}^3$$

$$M_p = F_y Z_p$$

$$= 250 \times 1895 = 474 \text{ kNmm}$$

$$P_y = A_s F_y$$

$$= 177.5 \times 250 = 44.4 \text{ kN}$$

Now find M_{pc} under P = 17.9 kN

Area of web required $= \frac{P}{F_y} = \frac{17.9 \times 10^3}{250} = 71.6 \text{ mm}^2$

Web depth $= \frac{P}{F_y \times t \times 2} = \frac{71.6}{1.56 \times 2} = 22.95$ mm

$$Z_{pc} = (30 \times 1.56 \times 14.22 \times 2 + (13.44 - 11.47)\ 1.56 \times 12.45 \times 4)$$

$$= 1484 \text{ mm}^3$$

$$M_{pc} = 250 \times 1484 = 371 \text{ kNmm} < 4\ 22$$

So even with the 1.3 kN concentrated load removed the chord is unsatisfactory. Shall determine what load can be supported.

Try reducing the moment and load by 8%.

$P_u = 16.5$ kN, $M_u = 388$ kNmm

Area for axial load = 66 mm^2

i.e. web depth = 21.1 mm

$$M_{pc} = (1331 + 2.86 \times 1.56 \times 12.0 \times 4)\ 250 = 386 \text{ kNmm} =$$

388 almost

Therefore can support an end reaction of

0.92 x 2.575 = 2.37 kN

before developing the first plastic hinge at G and this is summarized in Figure D8a. Then with the 1.3 kN Concentrated load back on the truss residual reaction = (2.58 - 2.37) + 1.04

= 1.25 kN

Now can continue to load the truss with a plastic hinge at G. This adjusted truss was then analysed by ACES.

On further loading the next plastic hinge forms at A and this results in the additional moments and forces of Figure D8b.

Check if hinge forms at A

$$P_u = \frac{10.8}{0.6} = 18.0 \text{ kN}$$

$$M_{pc} = (1331 + 1.90 \times 1.56 \times 12.49 \times 4)\ 250 = 370 \text{ kNmm}$$

$$M_p = \frac{222}{0.6} = 370 \text{ kNmm}$$

$M_p = M_{pc}$ so hinge is formed at A.

New residual reaction = 1.25 - 0.28 = 0.97 kN

Now can still further load the truss so that GH and AB act as cantilevers until plastic hinges form at B and H. Then will have four hinges and so collapse occurs. (See Figure D8c. Figure 6.8c is additional to Figures D8, (a and b)

Check hinge at H

$$M_u = \frac{120 + 11 + 91}{0.6} = 370 \text{ kNmm}$$

$$P_u = {}^{11.1}/0.6 = 18.5 \text{ kN}$$

From hinge at A only marginally overstressed so will accept hinge is formed at H

Check hinge at B

$$M_u = \frac{95 + 23 + 61}{0.6} = 298 \text{ kNmm}$$

$$P_u = \frac{10.8}{0.6} = 18.0 \text{ kN}$$

So hinge not yet formed at B. AB acts as a cantilever and requires an extra moment = (370 - 298) 0.6 = 43 kNmm

P_x 1200 = 43 so P = 0.04 kN

So total reaction to collapse this end panel

= 2.37 + 0.28 + 0.15 + 0.04

= 2.84 kN < 3.62 kN as required

Residual reaction = 0.78 kN

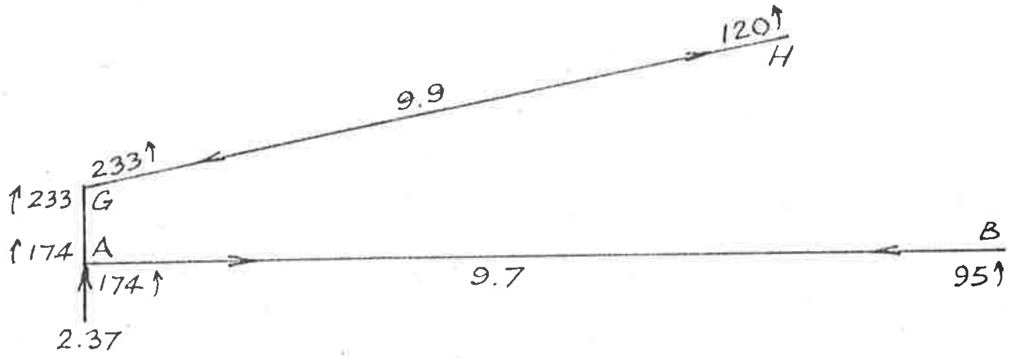


FIGURE D8a

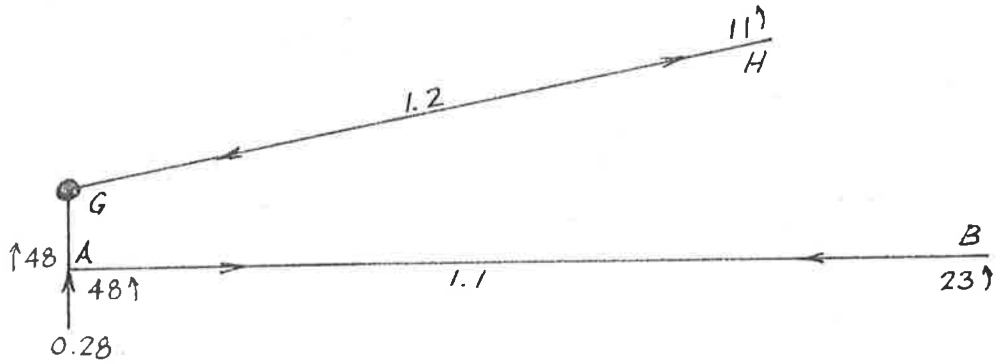


FIGURE D8b

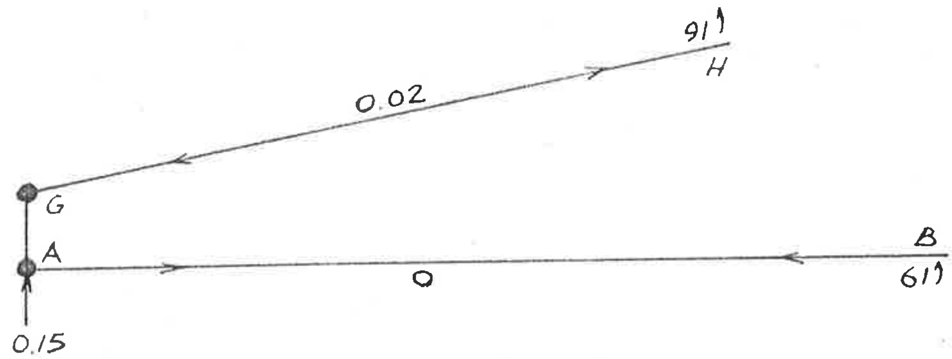


FIGURE D8c

Progressive Formation of Plastic Hinges

Other end of truss is just at point of collapse as reaction

there = 2.58 + 0.26

= 2.84 kN

Check the load required to cause collapse of this end panel by considering the sway mechanism (see Figure D2.)

Internal Work = External Work

$$M_p\ (\gamma + 6.31\gamma + 6.61\gamma + 1.3\ \gamma) = R.1200\gamma$$

$$15.22\ M_p = 1200R$$

$$R = \frac{2.84}{0.6} = 4.73\ \text{kN}$$

then $M_p = 373$ kNmm

For section using $F_y = 250$ MPa

Allowable $M_p = 474$ kNmm (see Page 51)

So for satisfactory as 373 < 474 but must now determine the reduction in allowable M_p due to the presence of axial loads.

Reactions

$$R_{VB} = \frac{2M_p}{1200} = 2 \times \frac{373}{1200} = 0.62\ \text{kN}$$

$$\sum H = 0 \text{ so } R_{VH} = 4.73 - 0.62 = 4.11\ \text{kN}$$

$$\sum M_B = 0$$

$$R_{HH} = \frac{1200R - 2M_p - R_{VH}.278}{214}$$

$$= (1200 \times 4.73 - 2 \times 373 - 4.11 \times 278)/214$$

$$= 17.70\ \text{kN} = -R_{HB}$$

Axial force in AB = 17.70 kN (tension)

" " " GH = $17.70 \times \frac{922}{938} + 4.11 \times \frac{172}{938} = 18.15$ kN (compression)

As both axial forces are almost the same then apply the same moment reduction to both chords and use the average force.

Web area required for axial load = $\frac{17.9}{.25}$ = 71.6 m^2

" depth " " " " = $\frac{71.6}{1.56 \times 2}$ = 22.9 m

Z_{pc} = (1331 + (13.44 - 11.45) 1.56 x 12.45 x 4)

= 1486 mm^3

M_{pc} = 372 kNmm against 373 kNmm as required. This then confirms the previous hinge by hinge progressive analysis.

Now before the trusses can fail they must also collapse the roof. The purlins are continuous over the full building length and the end trusses are only about half loaded compared to the interior trusses as they only support half of a bay. Also the back wall end truss is fully sheeted to ground thus strengthening it even further.

Therefore investigate the ability of the purlins to transfer load from the internal trusses to these end trusses.

Purlins

(a) Spanning over 3 bays each of 2.44m and under working load

BM_{max} = 2.44P kNm (simple beam span 2.44 x 3 = 7.32m with loads P at the third span points)

$BM_{allowable}$ = 17 x 1.5 x $\frac{24 \times 10^3}{10^6}$ = 612.10^{-3} kNm

P = $\frac{0.612}{2.44}$ = 0.25 kN

There are now six purlins so 1.5 kN total.

Also some positive moment occurs in the centre span from the roof load

W = DL + LL

w = 0.077 + .25 = .33 kN/m

BM = 0.25 x 0.33 x 2.44^2 = .500 kNm (Reference 23, Page 57)

P_{total} = 1.5 $\left(\frac{612-50}{612}\right)$ = 1.38 kN

$$\delta_{centre} = \frac{23P\ell^3}{648EI} = \frac{23 \times 0.23 \times 7.32^3 \times 10^{12}}{648 \times 14.10^3 \times 720.10^3} = 317 \text{ mm}$$

(Reference 23, Page 35)

For fixed at the ends

$$\delta = \frac{5P\ell^3}{648EI} = 69\text{mm} \quad \text{(Reference 23, Page 48)}$$

Further reduction results if composite action between purlins and the sheeting occurs.

Also clearly the two purlins on the far side of truss cannot assist very much as deflection there is much smaller. So the remaining four purlins need to support 0.78 kN

i.e. 0.20 kN each

Use composite section of sheeting plus purlins and convert to steel units and use 1m width.

$$40 \times \frac{14 \times 10^3}{200 \times 10^3} = 2.8\text{mm}$$

C. of G.

$$588y = 420 \times 0.21 + 168 \times 30.42$$

$$y = 8.84$$

$$MI = \frac{1}{12} \times 2.8 \times 60^3 + 168 \times 21.58^2 + 420 \times 8.63^2$$

$$= 160.10^3 \text{ mm}^4$$

Purlin alone $MI = \frac{1}{12} \times 2.8 \times 60^3 = 50.4 \times 10^3$

Using 0.20 kN/purlin, composite action, 50% fixity and deflection at the truss instead of the midspan of the beam

i.e. $\frac{20}{648}$ not $\frac{23}{648}$ for simply supported and $\frac{4}{648}$ not $\frac{5}{648}$

for fixed supports.

$$\delta_{truss} = \frac{75.5 + 15.1}{2} = 45.3 \text{ mm}$$

From the load test $\delta = 27$mm and so clearly the truss must have supported more of the load.

Now from material testing F_y = 280 MPa at least so will use this value.

Then because the strength of the end panel is directly proportional to the material yield strength

$$V_u = 2.84 \times \frac{280}{250} = 3.18$$

leaving 3.62 - 3.18 = 0.44

4 purlins → 0.11 kN each

$$\delta = 45.3 \times \frac{11}{20} = 24.9 \text{ mm}$$

This deflection now less than 27mm but will increase due to one purlin support (end truss at the open end) deflecting so now approximately correct.

For total load to be supported by the truss we need

$$F_y = \frac{3.62}{2.84} \times 250$$

$$= 319 \text{ MPa}$$

Top chord only checked so far as a beam so must now check it as a strut. (Reference 15, Section 10 and Reference 19).

$$P_{AC} = \frac{A_s F_{AC}}{0.6} \qquad \text{Use } F_y = 250 \text{ MPa}$$

$$= 177.5 \times \frac{89}{0.6} = 26.3 > 18.15 \text{ kN}$$

$$\frac{P}{P_y} = \frac{18.15}{44.4} = 0.41$$

$$\lambda = \frac{L}{\pi r} \sqrt{\frac{F_y}{E}} = \frac{940}{\pi . 11.63} \sqrt{\frac{250}{2.10^5}} = 0.91$$

Maximum value of

$$\frac{P}{P_y} = \frac{1 + \beta - \lambda}{1 + \beta + \lambda}$$

$$= \frac{1 + 1 - 0.91}{1 + 1 + 0.91} = 0.37 < 0.41$$

This indicates that the P-Δ effect causing some extra moment should not be neglected and that the maximum moment then occurs not at the ends as assumed but only near the ends.

Here eccentricity due to the moment is large at 21mm compared to $^D/_2$ of 15mm. Deflection under the axial load would surely be small so this extra moment from P-Δ effect would be small. Also note that the design moments are at the centrelines and not at the edge of the clear span so are slightly high thereby tending to cancel this extra moment.

Check stability

$$P \leq \frac{0.33\,\pi^2\,EI}{\ell^2}$$

$$\leq \frac{0.33\,\pi^2 \times 2.10^5 \times 24000}{10^3(1.13 \times 940)^2}$$

$$\leq 13.8 \text{ kN}$$

Applied P = 18.15 kN so unsatisfactory

Now this clause is to ensure that $F_{ob} > 3F_y$ as then no lateral instability effects apply. In other words no reduction to M_p for stability effects are necessary.

The $F_{ob} > 3F_y$ clause gives very nearly for an I section beam

$\frac{\ell}{r_y} \leq 960$ and then reduction to moment capacity (to rule 6.4 of AS 1250) is not applicable.

As this stability rule was derived for laterally unstable I sections and is not applicable to laterally stable box sections it will not therefore be used again in this review.

Lateral restraints

$$\text{Maximum spacing} = \frac{960 r_y}{\sqrt{250}} = 706 < 940 \text{ mm}$$

This restriction again is to ensure no moment reduction is applicable due to lateral instability.

As for stability this rule is not applicable here and so will not be used again in this review.

Diagonals

HB $\quad M = 42.8$ kNm (Figures D4 and D7).

$P = 0.2$ kN

Bending dominates

$$f_b = \frac{42.8 \times 10^3}{303} = 141 \text{ MPa} \quad \begin{array}{l} < 0.75 \times 250 \\ < 187 \text{ MPa} \end{array}$$

JC $\quad BM = 6.7$ kNm (Figures D4 and D7).

$P = 2.5$ kN compression

$$\ell/_r = 0.7 \times \frac{600}{3.52} = 119$$

$$F_{AC} = 59$$

$$f_{ac} = \frac{2.5}{.149} = 17 \text{ MPa}$$

$$f_b = \frac{6.7}{.303} = 22 \text{ MPa} \leftarrow \text{low}$$

Diagonals satisfactory

Columns

These are carrying axial load only

$P = 3.62$ kN

$$f_{ac} = \frac{3.62 \times 10^3}{2 \times 177.5} = 10.2 \text{ MPa}$$

Take a severe case for $\ell/_r$

$$\ell/_r = \frac{2700}{11.63} = 232$$

$F_{AC} = 18.0$ MPa satisfactory

Maximum $\ell/_r$ = 180 but satisfactory as there is restraint along the column length by the girts.

D4 Truss, DL - EWL

Under test loading failure occurred at the load case of location Adelaide, Wind Category 2, load factor 1.2 WL-DL and with an internal pressure coefficient of 0.4

Loads shown in Figure D9 are ultimate loads/bay for the combination of 1.2 WL-DL

$$DL = 0.42 \text{ kN/m}$$

$$= 2.50 \text{ kN total per truss}$$

WL

$$q_z = 0.6 \frac{(0.93 \times 42)^2}{10^3} = 0.915 \text{ kPa}$$

Roof $\quad C_p = 0.9 + 0.4 = 1.3$

$$W = C_p \times q_z \times S \times \ell \times \text{L.F.}$$

$$= 1.3 \times 0.915 \times 2.44 \times \frac{2.97}{0.984} \times 1.2 = 10.5 \text{ kN}$$

Wall $\quad C_p = 0.6 + 0.4 = 1.0$

$$W = 1.0 \times 0.915 \times 2.44 \times 2.65 \times 1.2 = 7.1 \text{ kN}$$

Frame then analysed under these loads using ACES

D4.1 Truss

First panel critical from DL + LL case. Ultimate reactions are shown on Figure D10.

$$V = (10.5 \times 0.984 - 1.25) \frac{2510}{2971} = 7.67 \text{ kN}$$

$$H = \frac{7.1}{2} = 3.55 \text{ kN (conservative - assumes pin base)}$$

$$M = H \times 53 = 188 \text{ kNmm}$$

Again use plastic theory on this end panel and so determine the load capacity of the truss. The collapse mechanism is shown in Figure D11.

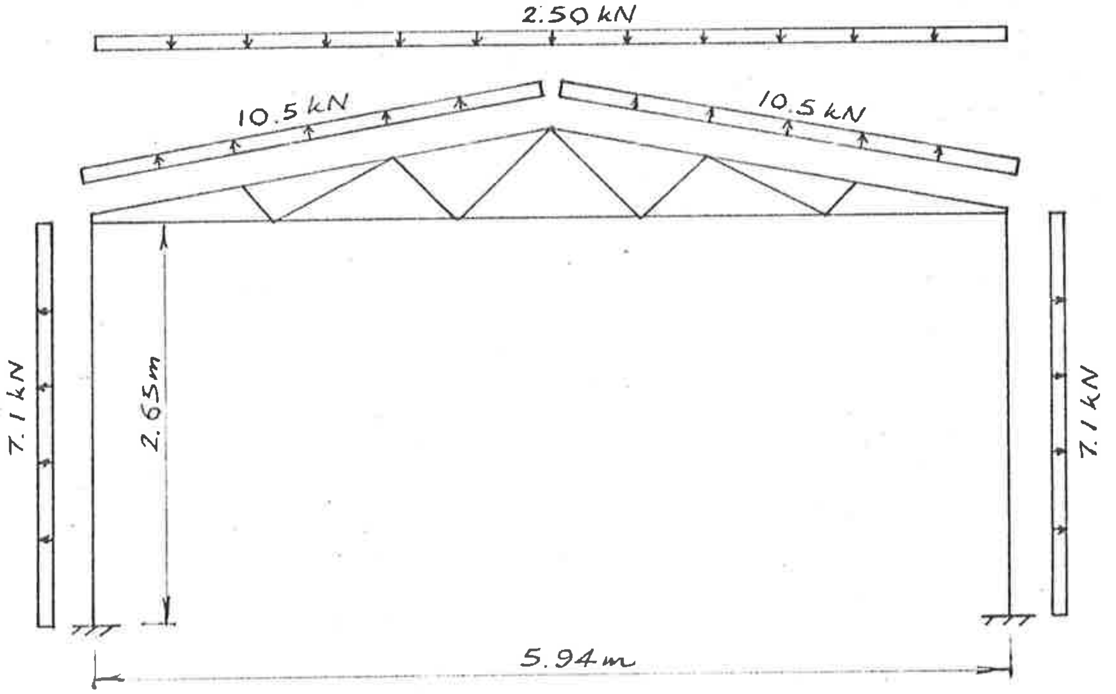


FIGURE D9

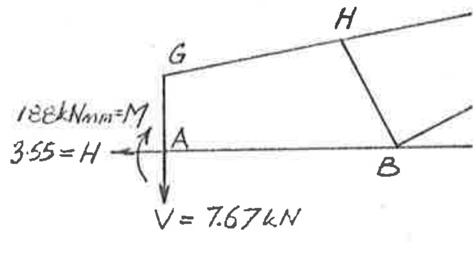


FIGURE D10

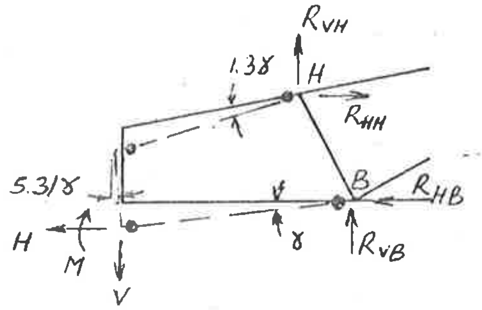


FIGURE D11

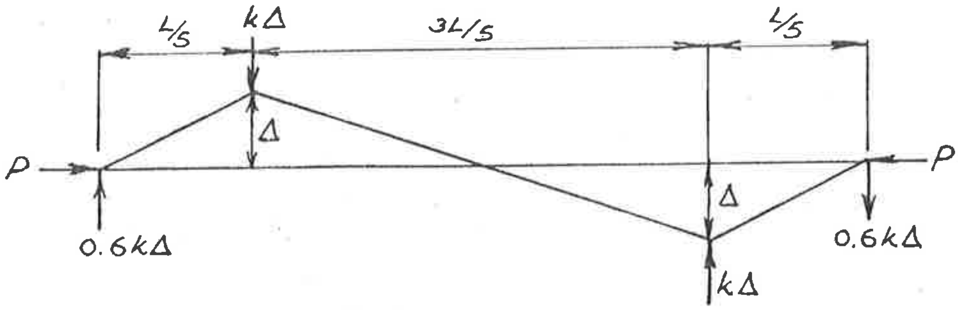


FIGURE 11a

Collapse Mechanism

Internal Work = External Work

$$M_p\,(\gamma + 6.31\gamma + 6.61\gamma + 1.3\gamma) = V \times 1200\gamma + M \times 5.31\gamma$$

$$15.22\,M_p = 1200V + 5.31\,M$$

$$= 1200 \times 7.67 + 5.31 \times 188 = 10{,}202$$

$$M_p = 670 \text{ kNmm}$$

Initially assume that no moment reduction results from the combination of axial load, moment then

$$M_p \text{ of section} = Z_p\,F_y$$

$$= 1895\,F_y = 670$$

$$\text{requires} \quad F_y = 354 \text{ MPa}$$

Try F_y = 360 MPa as this is the maximum yield value observed from the material testing (see Appendix A).

Clearly the truss will be unable to support all the load so assume that the truss will support the combination of 0.83V plus M and H.

$$M_p \text{ required} = \frac{0.83 \times 1200 \times 7.67 + 5.31 \times 188}{15.22}$$

$$= 568 \text{ kNmm}$$

Reactions

$$R_{YB} = \frac{2M_p - M}{1200} = \frac{2 \times 568 - 188}{1200} = 0.79 \text{ kN}$$

$$\sum V = 0 \qquad R_{VH} = 0.83 \times 7.67 - 0.79 = 5.58$$

$$\sum M_B = 0$$

$$R_{HH} = -R_{VH}\,\frac{278 - 2M_p + 0.83V.1200 + M}{214}$$

$$= (-5.58 \times 278 - 2 \times 568 + 0.83 \times 7.67 \times 1200 + 188)/214$$

$$= 24.02 \text{ kN}$$

$\sum H = 0$

$$R_{HB} = R_{HH} - H$$

$$= 24.02 - 3.55$$

$$= 20.47 \text{ kN}$$

As check calculate $\sum M_H$

$$M_H = -R_{VB}.278 + R_{HB}.214 - 0.83\ V.922 - M + 2M_p + H.214$$

$$= 0.79 \times 278 + 20.47 \times 214 - 0.83 \times 7.67 \times 922 - 188 + 1136 + 3.55 \times 214$$

$= -0.88 \neq 0$ but certainly checks satisfactorily.

Axial force in AB = 20.47 kN (compression)

" " " GH = $24.02 \times \frac{922}{938} + 5.58 \times \frac{172}{938}$

= 24.63 kN (tension)

So for AB with results for similar calculations for GH bracketed alongside AB

Area of web for axial load = $\frac{20.47}{.36}$ = 56.86mm^2 (68.42)

Depth " " $= \frac{56.86}{1.56 \times 2}$ = 18.22mm (21.93)

Z_{pc} = (1331 + (13.44 - 9.11)1.56 × 11.28 × 4)

= 1636 (1519)

M_{pc} = 589 (547)

Average M_{pc} = 568 as required

Then with the end panels at point of collapse

Residual Vertical reaction = 0.17 × 7.67

= 1.30 kN

So the purlins are required to transfer this load to the end trusses for the building to support the load. There are 3 purlins available so 0.43 kN/purlin

$$BM = 2.44 \times 0.43 + .025\ (1.19 - 0.08) \times 2.44^2 \times 1.2$$

$$= 1.05 + 0.20 = 1.25 \text{ kNm}$$

$$F_b = \frac{1.25 \times 10^6}{24 \times 10^3} = 52 \text{ MPa} \quad \text{(Ultimate)}$$

Seasoned Standard Grade Kapur

allowable bending stress = 17 MPa

Required Load Factor = $\frac{52}{17} = 3.06$

Reference 11 suggests that this is satisfactory.

Deflections are not considered as the load case is ultimate and therefore only necessary to show that the purlins have the bending strength to transfer the load to the end trusses. It must be ensured that the end trusses are not overloaded and this is satisfactory because

$$\frac{\text{load on end trusses}}{\text{load on internal trusses}} \approx \frac{0.5 + 0.17}{0.83}$$

$$\approx 0.81$$

The only other possible way of supporting some of the roof load was by the roof sheeting acting as cantilevers normal to the plane of the sheeting and so transferring some load to the side walls. As expected this provided negligible assistance.

D4.2 Bottom Chord, Strut Strength

Axial forces in the 5 panels immediately prior to buckling were

End Panels	20.5 kN
First Interior Panels	15.5 kN
Centre Panel	13.5 kN

These then were the ultimate panel forces.

Strut Strength

Use $F_y = 360$ MPa

Vertical buckling means buckling in the truss plane.

Horizontal buckling means buckling perpendicular to the truss plane.

a) End Panel

Vertical buckling

This panel was the only sway not prevented case.

$L = 1200$ mm, $G_A = 0$, $G_B = 0.9$, $\ell = 1.13L$, $\frac{\ell}{r} = 116$,

$F_{AC} = 72$ MPa

$$P_{AC} = \frac{A_s\ F_{AC}}{0.6}$$

$$= \frac{177.5 \times 72}{10^3 \times 0.6} = 21.3 \text{ kN} > 20.5 \text{ kN}$$

Horizontal buckling see (b) and (c) below

b) Horizontal buckling assuming no intermediate lateral restraints

$L = 5940$ mm, $\ell = L$, $\frac{\ell}{r} = 510$, $F_{AC} = 4$ MPa

$P_{AC} = 1.2$ kN < 17.1 kN (Mean force)

c) Horizontal buckling assuming effective lateral restraints existed at all four panel points

End Panel

Vertical buckling governs so from (a)

$P_{AC} = 21.3$ kN > 20.5 kN

First Interior Panel

$L = 1200$ mm, $G_A = 1.7$, $G_B = 1.3$, $\frac{\ell}{L} = 0.82$, $\frac{\ell}{r} = 85$,

$F_{AC} = 120$ MPa

$P_{AC} = 35.5$ kN > 15.5 kN

Centre Panel

$L = 1200$ mm, $G_A = G_B = 1.5$, $\frac{\ell}{L} = 0.82$, $\frac{\ell}{r} = 85$,

$F_{AC} = 120$ MPa

$P_{AC} = 35.5$ kN > 13.5 kN

d) Horizontal buckling assuming effective lateral restraints existed at the two outer panel points but no restraints existed at the two inner panel points.

i.e. L was 1200, 3600, 1200

So the centre panel at L = 3600 mm was critical

$G_A = G_B = 0.55$, $\frac{\ell}{L} = 0.7$, $\frac{\ell}{r} = 216$, $F_{AC} = 22$ MPa

$P_{AC} = 6.5$ kN < 14.5 kN

On observing the results of (b), (c) and (d) it was concluded that the correct analysis for horizontal buckling lay between (c) and (d). i.e. the truss diagonals provided full lateral restraint (to the bottom chord) at the outer panel points and partial lateral restraint at the inner panel points. That resulted in the buckling load of the centre panel being between 6.5 kN and 35.5 kN, namely 14.5 kN.

Lateral Restraints

Attempt by analysis to establish the amount of restraint afforded to the bottom chord by the truss diagonals. The diagonals could only provide restraint by cantilever action.

Restrained at all 4 Panel Points

Required restraint stiffness

$$k > \frac{18P}{L} \qquad \text{(Reference 19, Rule 3.3.4.4.)}$$

(Note: Not $\frac{4P}{\ell} = \frac{20P}{L}$ as loads are ultimate so then L.F. = 1.0)

For an individual cantilever restraint

$$\Delta = \frac{Fh^3}{3EI} \quad \text{so } k = \frac{3EI}{h^3}$$

Then for the maximum value of h

$$\frac{3EI}{h^3} = \frac{18P}{L}$$

$$h = \sqrt[3]{\frac{EIL}{GP}}$$

$$\doteq \sqrt[3]{\frac{2 \times 10^5 \times 1.85 \times 10^3 \times 6000}{6 \times 17100}} = 280 \text{ mm}$$

Outside Panel Points

$h_1 = 320$ mm, $h_2 = 760$ mm (2 diagonals at every panel point)

Equivalent single diagonal

$$\frac{1}{h^3} = \frac{1}{320^3} + \frac{1}{760^3}$$

$h = 312$ say 310 mm > 280 mm

Inside Panel Points

$h_1 = 560$ mm, $h_2 = 800$ mm

so $h = 507$ say 510 mm > 280 mm

This confirms that the bottom chord was not fully restrained at all 4 panel points so

$P_{AC} < 35.5$ kN

Restrained only at the Outside Panel Points

For Stability

$\Sigma M_B = 0$ (See Figure D11a)

$$0.6\, k\, \Delta\, \frac{L}{5} > P\Delta$$

$$K > \frac{8.33P}{L}$$

i.e. $$\frac{3EI}{h^3} > \frac{8.33P}{L}$$

$$h < \sqrt[3]{\frac{EIL}{2.78P}}$$

$$< 360 \text{ mm}$$

actual $h = 310$ mm < 360 mm

This buckling case was therefore satisfied. However, this only gave $P_{AC} = 6.5$ kN < 14.5 kN (buckling force)

Note:

Any rotation of the top chord under torsional moment (diagonal end moment) would reduce the diagonal stiffness. However, any such rotation would have been counteracted by

a) The purlin could have relieved the top chord of some of the diagonal end moment so reducing the torsional moment on the top chord.

b) The bottom chord would also have rotated in order to equalise its slope with the diagonal's end slope. The moment causing the bottom chord rotation would also in turn apply to the diagonal and so would in fact reduce the diagonal moment. Thus this effect actually increases the diagonal stiffness.

Checking Diagonal Stresses resulting from the Lateral Restraint Force

$$\text{Force} = 0.25 \times \frac{17.1}{2} = 0.21 \text{ kN}$$

$$\text{BM} = 0.21 \times 310 = 65 \text{ kN mm}$$

$$f_b = 143 \text{ MPa} < 280 \text{ MPa}$$

Adding the stresses resulting from the diagonals other function (as a truss member) would certainly leave the member severely stressed.

However, this restraint force could be high because

a) Have two partial restraints besides the two full restraints so that could reduce the force on the two full restraints.

b) There exists evidence suggesting that using the lateral force as 2½% of the longitudinal force was conservative.

$$\Delta = \frac{P l^3}{3EI} = \frac{210 \times 310^3}{3 \times 2 \times 10^5 \times 1.85 = 10^3} = 5.6 \text{ mm}$$

Torsion stress on the top chord assuming that all of the diagonal moment was taken by the truss top chord

$$f_t = \frac{T}{2\bar{A}t}$$

$$= \frac{65 \times 10^3}{2(30-1.6)^2 \times 1.6} = 25.2 \text{ MPa}$$

This calculation, despite using a conservative assumption for the torsion moment still gave a low stress.

Overall, the analysis has reasonably confirmed that the buckling strength was at least 6.5 kN. The analysis that resulted in a buckling strength of 6.5 kN was based on full lateral restraint at the outer panel points but without any lateral restraint at the inner panel points. In actual fact, there were partial lateral restraints at the inner panel points and so neglecting these restraints was surely a conservative assumption.

The outer panel points were quite stiff and besides providing full lateral support for the 6.5 kN buckling load case they also almost provided their part of the total stiffness requirement for the 35.5 kN buckling load case.

Also it was worth noting that 14.5 kN was much closer to 6.5 kN than 35.5 kN and in fact was 0.28 of the difference of 29 kN between the two buckling loads.

Therefore, it is the author's opinion that the effect of including these partial restraints at the inner panel points would be the raising of the buckling load from 6.5 kN to 14.5 kN.

The analysis has not completely confirmed that the buckling load was 14.5 kN and so the load test alone provides the only absolute confirmation.

D5 Column, DL - EWL

D5.1 Pin Jointed at Top, Fixed at Base

Ultimate Loads - Distributed 7.1 kN total

Axial 10.5 - 1.25 = 9.25 kN

Elastic Analysis by ACES (see Figure D12) with resulting moments and forces taken as the ultimate moments and forces. i.e. no redistribution due to plasticity.

Load test result, see Figure D14.

Try F_y = 280 MPa

Main Vertical Chords

Outside Column Leg at Top, see Figures D15 and D16.

$$BM_{38mm} = 572 - \frac{(572 + 139)\ 38}{1056} - \frac{549 \times 38}{477} = 503 \text{ kNmm}$$

Have 503 kNmm with 16.9 kN

Area required for the axial load $= \frac{16.9}{.28} = 60 \text{ mm}^2$

i.e. 19.23mm of web

$$Z_p = 1331 + (13.44 - 9.61)1.56 \times 11.52 \times 4$$

$$= 1331 + 275 = 1606$$

$$M_p = 1606 \times 0.28 = 450 \text{ kNmm} < 503 \text{ kNmm}$$

Overloaded by 12%

Inside Column Leg at Base

BM = 553 kNm, P = 10.14 kN

Area required for the axial load $= \frac{10.14}{.28} = 36.2 \text{ mm}^2$

i.e. 11.61 mm of web

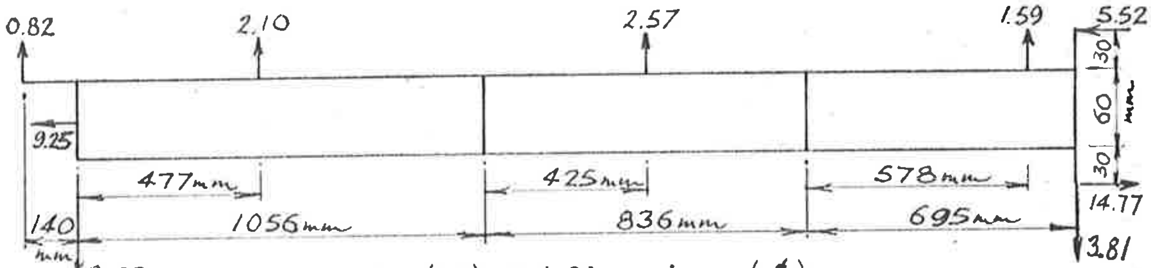
$$Z_p = 1331 + (13.44 - 5.80)\ 1.56 \times 9.61 \times 4 = 1790$$

M_p = 501 kNmm < 553 kNmm Overloaded by 10%.

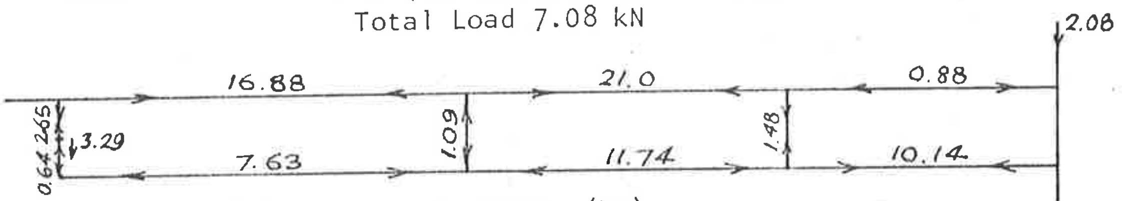
Inserts between the main chords

$$BM = 683 - (683 + 630)\ \frac{15}{60}$$

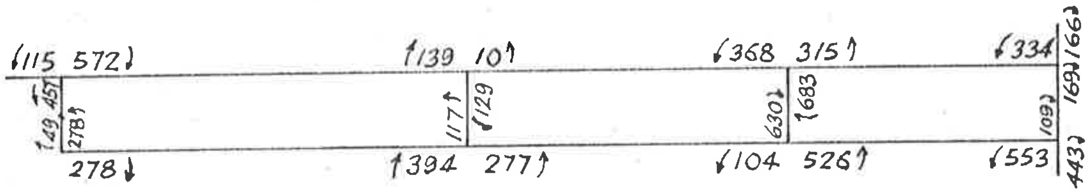
$$= 355 \text{ kNmm}$$



LOADS (kN) and Dimensions (℄)
Total Load 7.08 kN



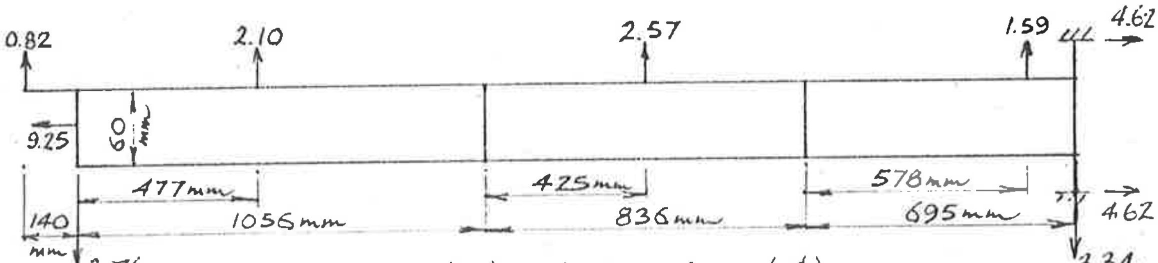
AXIAL FORCE (kN)



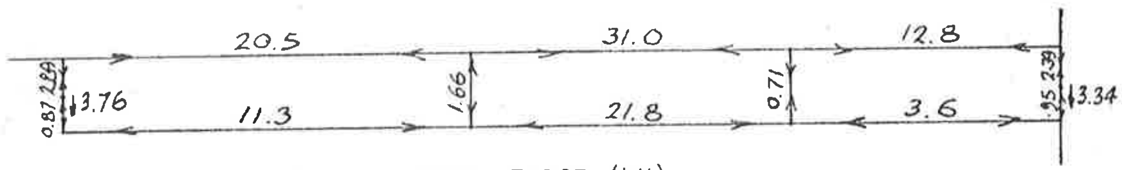
MOMENT (kNmm) ℄

Results from ACES (Pinned Top, Fixed Base)

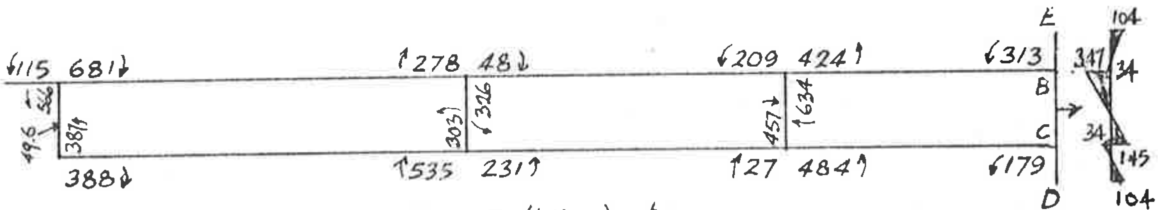
FIGURE D12



LOADS (kN) and Dimensions (℄)
Total Load 7.08 kN



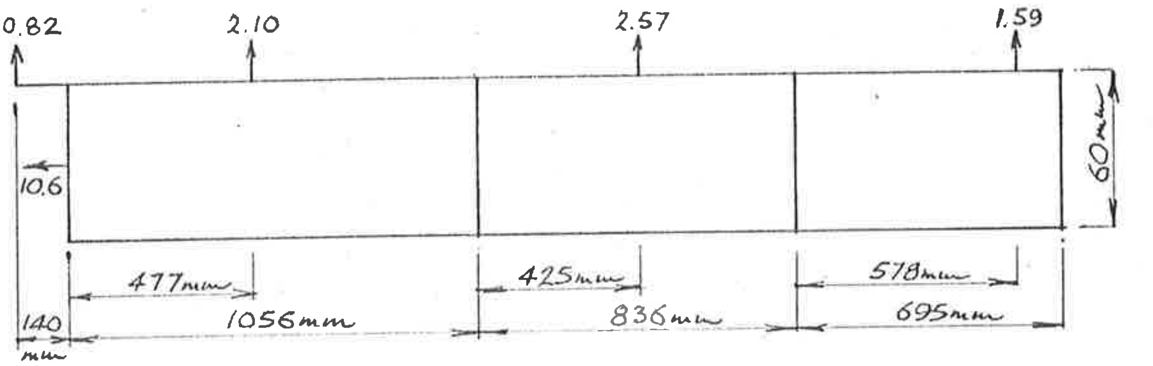
AXIAL FORCE (kN)



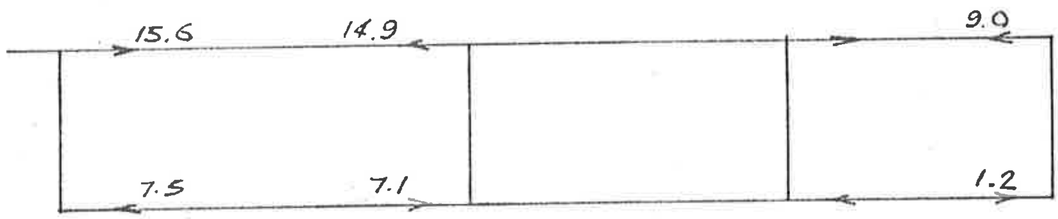
MOMENT (kNmm) ℄

Results from ACES (Pinned Top and Base)

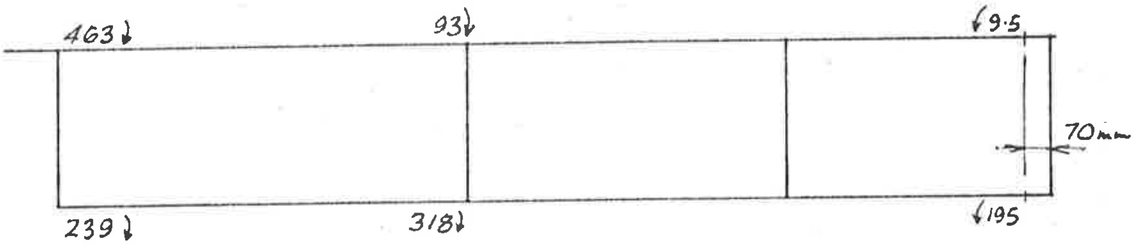
FIGURE D13



LOADS (kN)



AXIAL FORCES (kN)



MOMENTS (kNmm)

Results from Load Test - Assembled Garage, Fixed Base

FIGURE D14

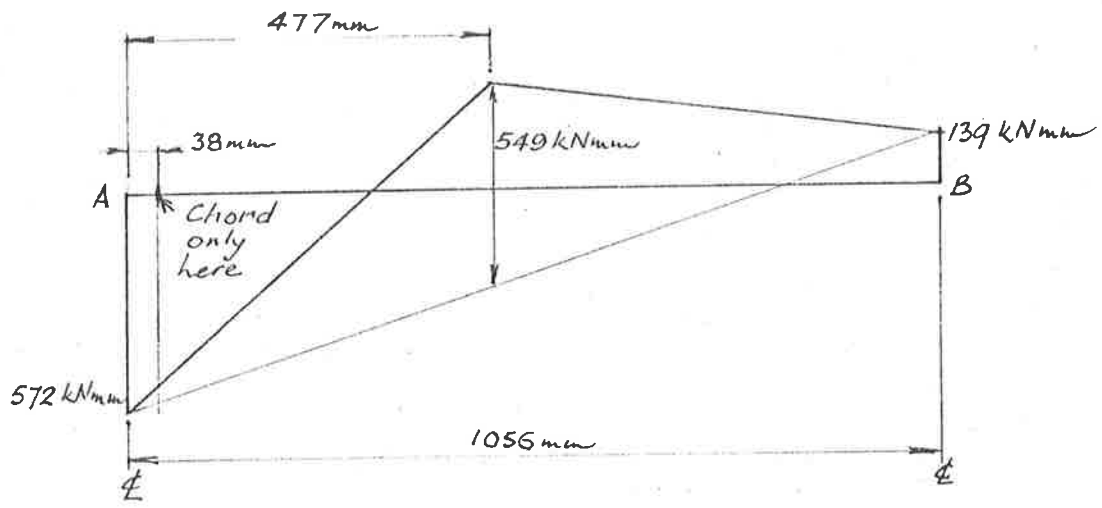


FIGURE D15

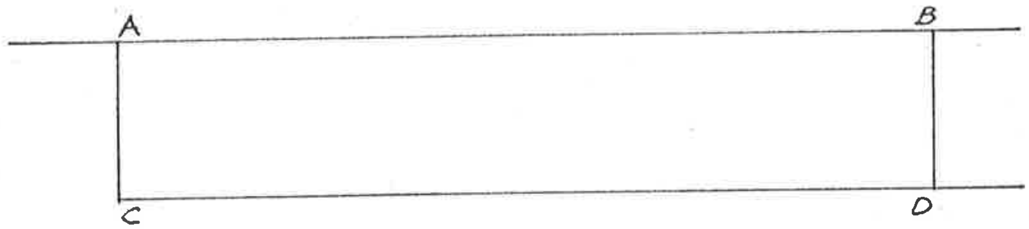


FIGURE D16

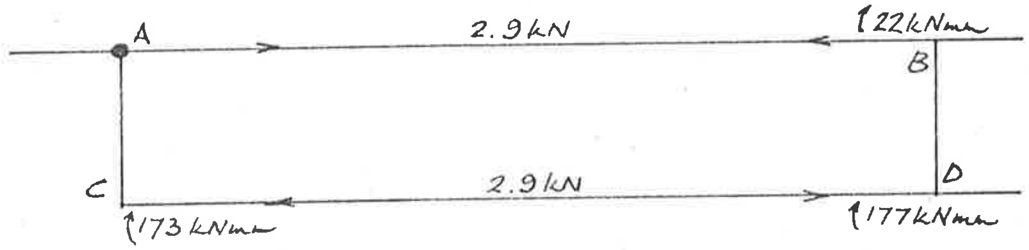


FIGURE D17

Area = $2 \times d \times t$

$= 2 \times 77 \times 1.56 = 240\ mm^2$

$Z_p = 2\ \frac{td^2}{4} = 1.56 \times \frac{77^2}{2} = 4625\ mm^3$

$M_p = 1295\ kNmm > 355\ kNmm$ very safe

Base Plate

$BM = 443 \times \frac{15}{30} = 221\ kNmm$

$Z_p = 40 \times \frac{6^2}{4} = 360\ mm^3$

$M_p = 101\ kNmm$

Overloaded by 119%

Bolt also overloaded

14.8 kN on 8mm Structural Bolt

Yield Load 8.8 kN using $F_{y_{min}} = 240$ MPa

Ultimate Load 14.6 kN using $F_{min} = 400$ MPa

so well overloaded.

As baseplate and bolt both yield the fixed base reverts to a pinned base and the stresses at the base are relieved. Next examine the behaviour with a pinned base.

D5.2 Pin Jointed at Top and Bottom

(Loads as for the Fixed Base Case, Analysis by ACES (see Figure D13.)

Base Plate

$BM = 347 - \frac{(347 + 145)15}{60} = 224\ kNmm$

so still overloaded by 122%.

However unlikely to fail at the edge of the section due to 3mm fillet weld at that edge so at 3mm from edge of section.

$BM = 347 - 492 \times \frac{18}{60} = 199\ kNm$

still overloaded by 97%

By Ultimate Load Theory the column moments can be redistributed provided that the total sway is still satisfied, therefore make

$$BM_{BC} = BM_{CB} = (313 + 179)/_2 = 246$$

So in BC

$$BM_{max} = 246 - \frac{492 \times 18}{60}$$

$$= 99 \text{ kNmm} < 101 \text{ kNmm}$$

Other moments in the base plate greater than 99 kNmm adjusted using the same argument making

$$BM_{DC} = BM_{EB} = 99$$

so $$BM_{BE} = BM_{CD} = 39$$

$$BM_{max} = 99 \text{ kNmm} \qquad \text{therefore safe}$$

Main Chords

Top Outside Panel is critical

$$BM_{38mm} = 681 - \frac{(681 + 278)}{1056} \times 38 - 549 \times \frac{38}{477}$$

$$= 603 \text{ kNmm with } P = 20.5 \text{ kN}$$

$$Z_p = 1470$$

$$M_p = 412 < 603$$

overloaded by 46%

D5.3 <u>Fixed at Top with Pinned or Fixed at Base</u>

These cases produced results even further removed from the test results than the two previous cases. There was considerable play in the column to truss connection rendering top fixity most unlikely and therefore these cases were dismissed.

(See Appendix E - Drawing DG3).

D5.4 Load transfer via the girts

Maximum load transfer per girt

$$BM_{max} = P\ell + .025\ W\ell = fb \times Zp$$

$$= 2.44P + 0.025 \times \frac{7.1}{3} \times 2.44 = (17 \times 3) \times (16 \times 10^3) / 10^6$$

$P = 0.24$ kN

Ultimate Load Transfer per column

$= 0.24 \times 2$

$= 0.48$ kN

Total Load = 7.1 kN i.e. only 7%

Now investigate deflection.

Strengthen MI for deflection by considering the composite action on a 1m wide strip.

Convert to steel units

$$b(\text{steel}) = b(\text{timber}) \times \frac{E(\text{timber})}{E(\text{steel})}$$

$$60 \times \frac{14}{200} = 4.2\text{mm}$$

C of G

$$588y = 420 \times 0.21 + 168 \times 20.42$$

$$y = 5.98\text{mm}$$

$$MI = \frac{1}{12} \times 4.2 \times 40^3 + 168 \times 14.44^2 + 420 \times 5.77^2 = 71{,}400\text{mm}^4$$

Purlin alone MI = 22400

ratio 0.31

Using 0.24 kN per purlin, composite action, 50% fixity and deflection at truss, not at midspan.

Simply Supported

$$\delta_{ss} = \frac{20}{648} \times \frac{0.24 \times 7.32^3 \times 10^{12}}{14.10^3 \times 320.10^3} \times 0.31$$

$$= 200 \text{ mm}$$

Fixed

$$\delta_F = \frac{5}{20} \times 200 = 50\text{mm}$$

$$\text{so } \delta = \frac{200 + 50}{2} = 125\text{m}$$

Now column alone under applied loads deflected about 30mm

Purlin contribution $= \frac{30}{125} \times 0.24 = 0.06$ kN

Negligible assistance so will discard.

D5.5 Load Transfer via the sheeting

The next consideration was P - Δ effect resulting from the curvature of the sheeting caused by the column deflecting.

This placed the sheeting in tension so resulting in a resisting moment.

$$\delta = R(1 - \cos\theta)$$

$$\delta = 30\text{mm}$$

$$30 = R(1 - \cos\theta)$$

$$\sin\theta = \frac{1293.5}{R}$$

Solving $\theta = 2^{o}42'$, $R = 27{,}457$mm

Arc length $= 2\theta \times R$

$= 2587.55$mm

i.e. extension = 0.55mm

This length change is minute and can easily be lost by sheet slippage over nails and by nails pressing into the girts.

This assistance therefore neglected.

Finally composite action between the column and the sheeting was considered but was of no use as the column was not a truss so shear between the panels was transferred via bending in the column legs instead of axial loads in the bracing and the sheeting has no transverse bending strength.

Therefore the column alone must support all of the load.

D5.6 Collapse Mechanisms

So far this review has essentially used an elastic analysis (ACES) but with the ultimate loads on the column. The columns have been shown to be unsatisfactory for both pinned and fixed bases under this elastic analysis.

Therefore now try redistributing the moments using Plastic Theory and as the column is clearly much closer to a pinned base than a fixed base this review will only consider the pinned base case.

Top Panel - Sway Mechanism (governing mechanism, see below)

Hinges form in the column at the edges of the inserts between the main chords so panel length is the centre to centre distance minus two half insert lengths (2 x 38 = 76mm).

Internal Work = External Work (See Figure D13).

$$4M_p = (3.76 - 0.82) \times (1056 - 76) - 2.10\ (1056-477-38)$$

$$= 1745$$

$$M_p = 436 \text{ kNmm}$$

Axial loads in legs are 20.5 kN (tension) and 11.3 kN (compression) and for these loads on the actual section, using F_y = 280 MPa.

$$M_{pc} = 410 \text{ kNmm and } 494 \text{ kNmm}$$

Average M_{pc} = 452 kNmm > 436 kNmm

Beam and Combined Mechanisms were also considered but were less severe than the Sidesway Mechanism.

M_p = 253 kNmm for the Beam Mechanism

M_p = 386 kNmm for the Combined Mechanism

Bottom Panel - Sway Mechanism (governing mechanism)

Hinges at base form in the Base Plate and so limit moments here to 246 + 39 = 285 kNm in the outside leg and 246-39 = 207 kNmm in the inside leg.

Internal Work = External Work

$2\ M_p + 285 + 207 = 3.34 \times (695 - 38) - 1.59\ (578-38)$

M_p = 421 kNmm

Axial loads are 12.8 kN (tension) and 3.6 kN (compression) and so for these loads, using F_y = 280 MPa

M_{pc} = 482 kNmm and 527 kNmm

Average M_{pc} = 504 kNmm > 421 kNmm

Centre Panel - Beam Mechanism (governing mechanism)

Internal Work = External Work

$4M_p = 2.57 \times 380$

M_p = 244 kNmm

Even though axial loads in this panel are the highest for the column this panel obviously was satisfactory.

Therefore column was satisfactory for a pinned base and for the ultimate loads as shown on Figure D13.

Checking strut action to AS 1250 Section 10

Centre Panel

$$\ell/_L = \frac{1.3 \times 836}{11.63} = 93 \rightarrow F_{AC} = 93\text{MPa}$$

$$P_{AC} = 177.5 \times \frac{93}{0.6}$$

$$= 27.5 \text{ kN} > 21.8 \text{ kN}$$

Top Panel

$$\ell/_L = 1.1 \times \frac{1056}{11.63} = 100 \rightarrow F_{AC} = 83 \text{ MPa}$$

$$P_{AC} = 177.5 \times \frac{83}{0.6} = 24.5 \text{ kN} > 11.3 \text{ kN}$$

$$\frac{P}{P_y} = \frac{11.3}{49.7} = 0.23$$

$$\lambda = \frac{1056}{\pi \times 11.63} \sqrt{\frac{280}{2.10^5}} = 1.08$$

$$\frac{P}{P_y} < \frac{0.92}{3.08}$$

$$< 0.30 \qquad \text{satisfactory}$$

D6 Garage, DL + SWL

D6.1 General

The maximum test case was Category 2, Load Factor 1.2, Internal Pressure = Internal Suction = 0. On comparing the case with the same load case for DL + EWL, it was found from the strain gauge readings that DL + SWL was the less severe of the two cases.

This was expected as DL + SWL had less vertical load than DL + EWL and although DL + SWL had considerable lateral load against zero lateral load for DL + EWL, most of that lateral load was carried by the sheeting so making DL + EWL overall a more severe case than DL + SWL. This review thus concentrates on the effect on the building of the lateral load. Assuming all the column loads were equally shared between the top and the bottom of the column.

$$\text{Total ultimate lateral load} = \tfrac{1}{2}\text{ Wall load} - \text{Roof lateral load}$$

$$= (17.1 + 11.1)/2 - (21.3 - 16.06)\ 0.18$$

$$= 13.3 \text{ kN}$$

There were three bays and so the roof sheeting received load from $2\frac{1}{2}$ bays with the last $\frac{1}{2}$ bay load applied directly to the end wall.

Therefore if all the lateral load was taken by the roof

$$V = 13.3 \times \frac{2.5}{3} = 11.1 \text{ kN}$$

Deflections were noted for both the bare frame and the clad frame. The four roof trusses were numbered in order from T1 at the front of the building to T4 at the back of the building. T2 and T3 were the internal trusses with T2 being the closest to the open front of the building.

Deflections (mm)

Total Column Load (kN)	Single Bare Frame with Knee Braces		Clad Frame without Knee Braces, Truss T2(T3)	
	Base Pinned	Base Fixed	Base Pinned	Base Fixed
1.0	90			
1.9		76		
9.4			43 (28)	47 (40)

Ratio of $\frac{\text{Clad Frame Deflection}}{\text{Bare Frame Deflection}}$

Pinned Bases 0.05 (.034)

Fixed Bases 0.125 (0.106)

Note: (1) The Fixed Base case was discarded from any further investigations as it was considered that the Pinned Base case gave the most reliable set of deflections. This was because the Pinned Base loads were applied before the Fixed Base loads and considerable permanent deflections (8 to 19 mm) resulted from the Pinned Base loads. The structure was restored to verticality by simply pushing it back before the Fixed Base loads were applied. The resulting structure slackness resulted in higher deflections than those simply attributable to the applied loads and so the Fixed Base case was not further considered. See Appendix C and 5.8.2 for further explanation.

(2) The bare frame tests had the load on one column only whereas the clad frame had loads on both the roof and the walls.

(3) Bare frame tests had knee braces at the column tops whereas the clad frame was finally tested with the knee braces disconnected. Therefore, for a true comparison it is the author's considered opinion that the radios should be further downgraded.

(4) At the maximum test load it was noted that some of the roof nails in the roof bay adjacent to the end wall were bent over so indicating that the roof was close to its ultimate load state. The roof sheeting and the roof nails did not, however, appear to be sufficiently distressed to indicate total failure.

(5) Overall, it appeared reasonable to assume that almost all of the lateral load was carried by the sheeting and therefore the next part of the review was based on the hypothesis that the sheeting supported the total lateral load.

D6.2 Roof Strength

A single roof panel 2.44 mm x 3.0 m failed at a load of 4.0 kN when loaded as a cantilever of 2.44 m span. The actual roof had two 2.44 x 3.0 m panels per bay so the anticipated roof strength with regards to the sheeting was 8.0 kN.

The actual building was 3 bays long (7.32 m). Assuming that the sheeting supported all of the lateral (shear) load the truss loads were:

T1, T4 $\quad V = \frac{13.3}{6} = 2.22$ kN

T2, T3 $\quad V = \frac{13.3}{3} = 4.44$ kN

Resulting in the total effects

Shear = 2.22 + 2 x 4.44 = 11.1 kN

Complementary Shear = (2.22 x 7.32 + 4.44 (4.88 + 2.44))/5.94
= 8.21 kN

This then is illustrated in Figure D20.

The test panel (Figure D18) resulted in the comparable forces of

Shear = 8.0 kN

Complementary Shear = 3.25 kN

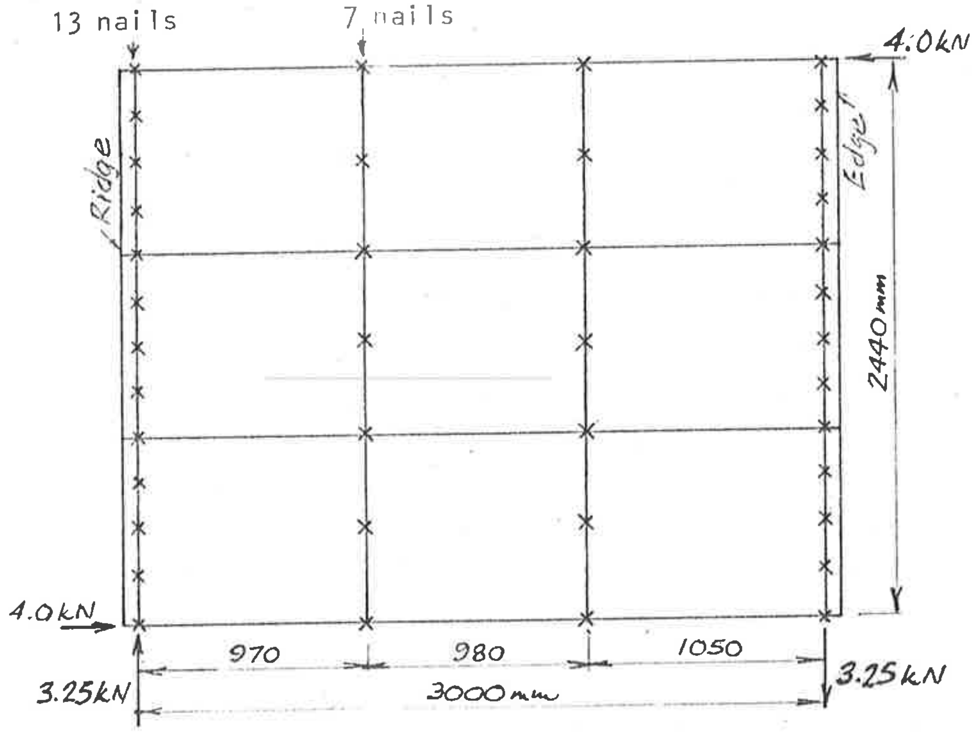
Now on the building the bottom purlin and the top girt were in fact the same member (Drawing DG2). This meant that the complementary shear did not in fact, accumulate along this purlin but instead was progressively transferred to the side walls via the girt (purlin) fixings. Thus the worst roof panel had

Shear = 11.1 kN

Complementary Shear = 11.1 x 2.44/5.94 = 4.56 kN

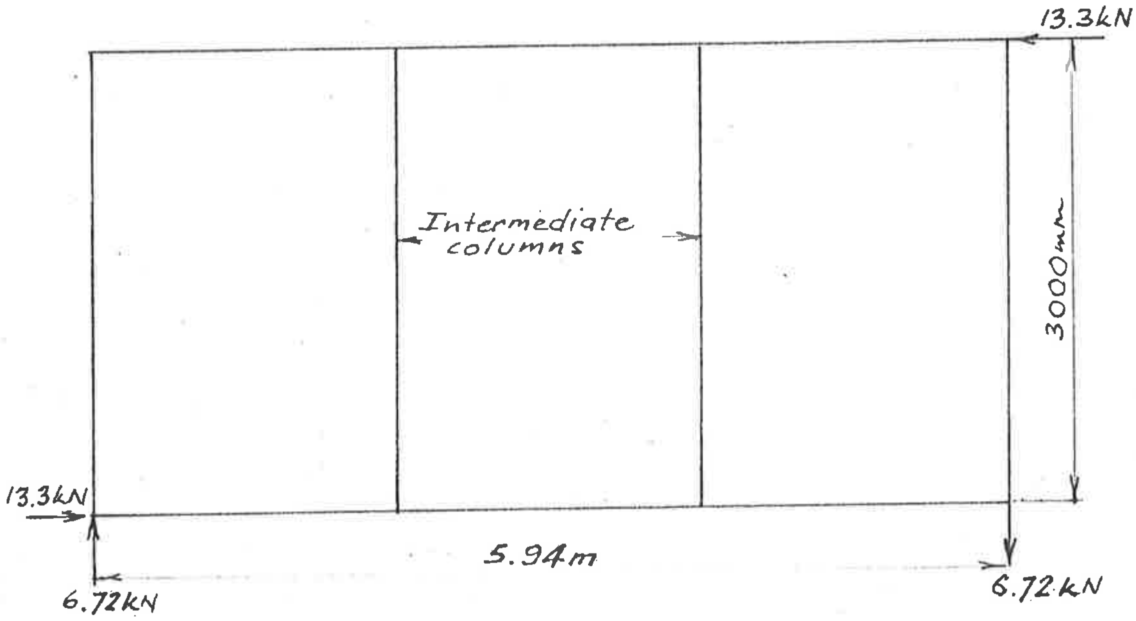
Even this complementary shear was not the true girt force as it was progressively transferred to the side walls throughout each panel length via the girt fixings. However, by taking a reduced lever arm, although that reduces the complementary shear, it did not reduce the force on each individual fastener. This was because the complementary shear had to be transferred via the roof fasteners to the wall fasteners and then the force was simply proportional to the fastener spacing.

Therefore, if the roof did indeed support all of the lateral load then the actual roof proved to be stronger than indicated by the test panel. Now this may have been true but surely could not be assumed so and therefore the hypothesis of the sheeting



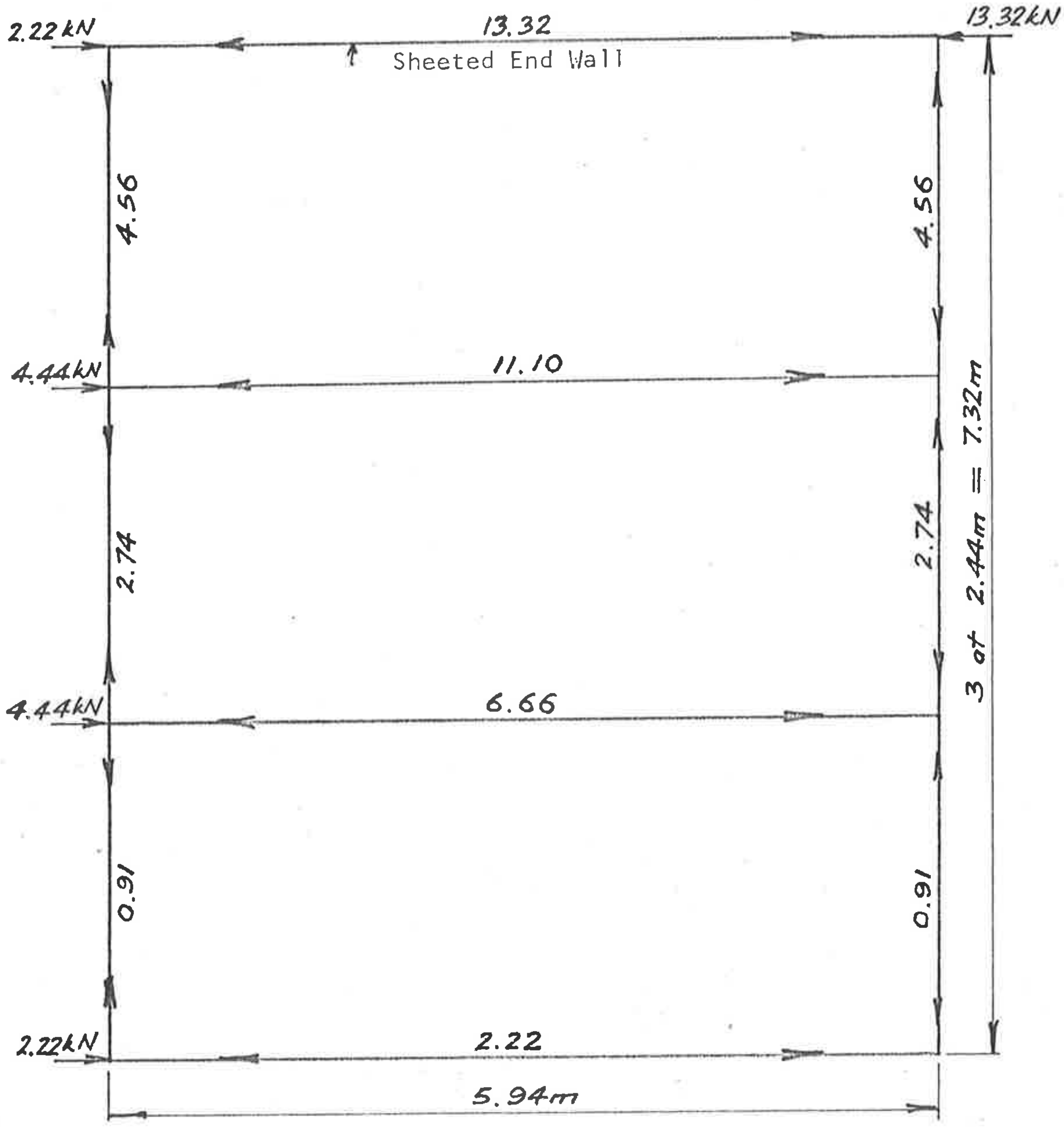
3 sheets, 4 purlins
x → nailing pattern
Roof Panel

FIGURE D18



End Wall

FIGURE D19



Roof Plan

FIGURE D20

supporting all the lateral load should perhaps be revised.

The test panel deflected 7.0 mm at 4.0 kN shear load (Figure 4.12) whereas the differential deflection between the end truss T4 and the adjacent truss T3 at the maximum total lateral load of 13.3 kN was 9.0 mm (Figure C3b). That then could have been taken as indicating that the end panel was at its ultimate load and that some of its load was, in fact, taken by the frame. Against this statement, however, is statement 4 of statements 1 to 6 of D6.1.

D6.3 Moment Capacity of Truss to Column Connection

Reference drawing DG 3

Shear Capacity

$$V = \frac{Fy}{\sqrt{3}} \times \quad (d - 10) \times t \times 2$$

$$= \frac{360}{\sqrt{3}} \times 15 \times 1.6 \times \frac{2}{10^3} = 10.0 \text{ kN}$$

Bearing Area

$$A = \frac{V}{Fy}$$

$$= \frac{10000}{360} = 27.8 \text{ mm}^2$$

Bearing Length

$$L = \frac{A}{b}$$

$$= \frac{27.8}{25} = 1.11 \text{ mm}$$

$$\text{Moment Capacity} = 10.0 \times (75 - 1.11)$$

$$= 739 \text{ kNmm}$$

Resulting Frame Shear

$$V = \frac{2 \times 739}{2650 - 38} = 0.57 \text{ kN}$$

Therefore, even allowing all 3 frames the resulting shear reduction is 3 x 0.57 = 1.7 kN.

Thence making the end panel shear

= 11.1 - 1.7

= 9.4 > 8.0 kN

Therefore, it does appear likely that the roof did in fact support more shear load than expected. Worth noting again that the frames had true pinned bases and therefore no possible shear transfer through the bases.

D6.4 Side Wall Strength

The side walls deflected about 10 mm longitudinally under side wind loading, on the building but they deflected in opposite directions, indicating that the side walls supported equal but opposite longitudinal forces. That supports the hypothesis that the complementary shear loading resulting from the roof shear was taken by the side walls. Another strong point in favour of that hypothesis is that if the side walls did not support this complementary shear then the shear loads would have had to be taken by the frames as there was simply nothing else available to support the complementary shear.

Shear load = 8.21 kN

Wall strength

Wall panel as tested took 5.33 kN (Figure 4.17)

Wall has 3 panels so strength = 16.0 kN >8.21 kN

D6.5 End Wall Strength (Figure D19)

Shear = 13.3 kN

Complementary shear = 6.72 kN

Comparative figures for the wall panel as tested

Shear 5.33 kN

Complementary shear = 5.33 x 2.7/2.44 = 5.90 kN

Adjusting for the length radio

$$\text{Shear} = 5.33 \times \frac{5.94}{2.44} = 13.0 < 13.3 \text{ kN}$$

Complementary shear = 5.90 < 6.72

The end wall had 5 girts compared to 4 for the test panel and so the

$$\text{Complementary shear/fastener} = \frac{6.72}{5} = 1.34 < \frac{5.90}{4} = 1.48$$

Therefore, it was observed that the end wall was marginally overstressed and needed the steel frame to support the small balance (0.3 kN) of the total lateral load. That would surely seem to be possible and in any case, the end wall seems stronger than the roof. Overall, the roof appears to be the weakest component with regards to supporting the lateral load.

D6.6 Tensile plus Shear Loads on the Nails

This was investigated but it was found that after the gain resulting from vector summing that the effect on the nail shear load capacity due to concurrent tensile load was small and could therefore be neglected.

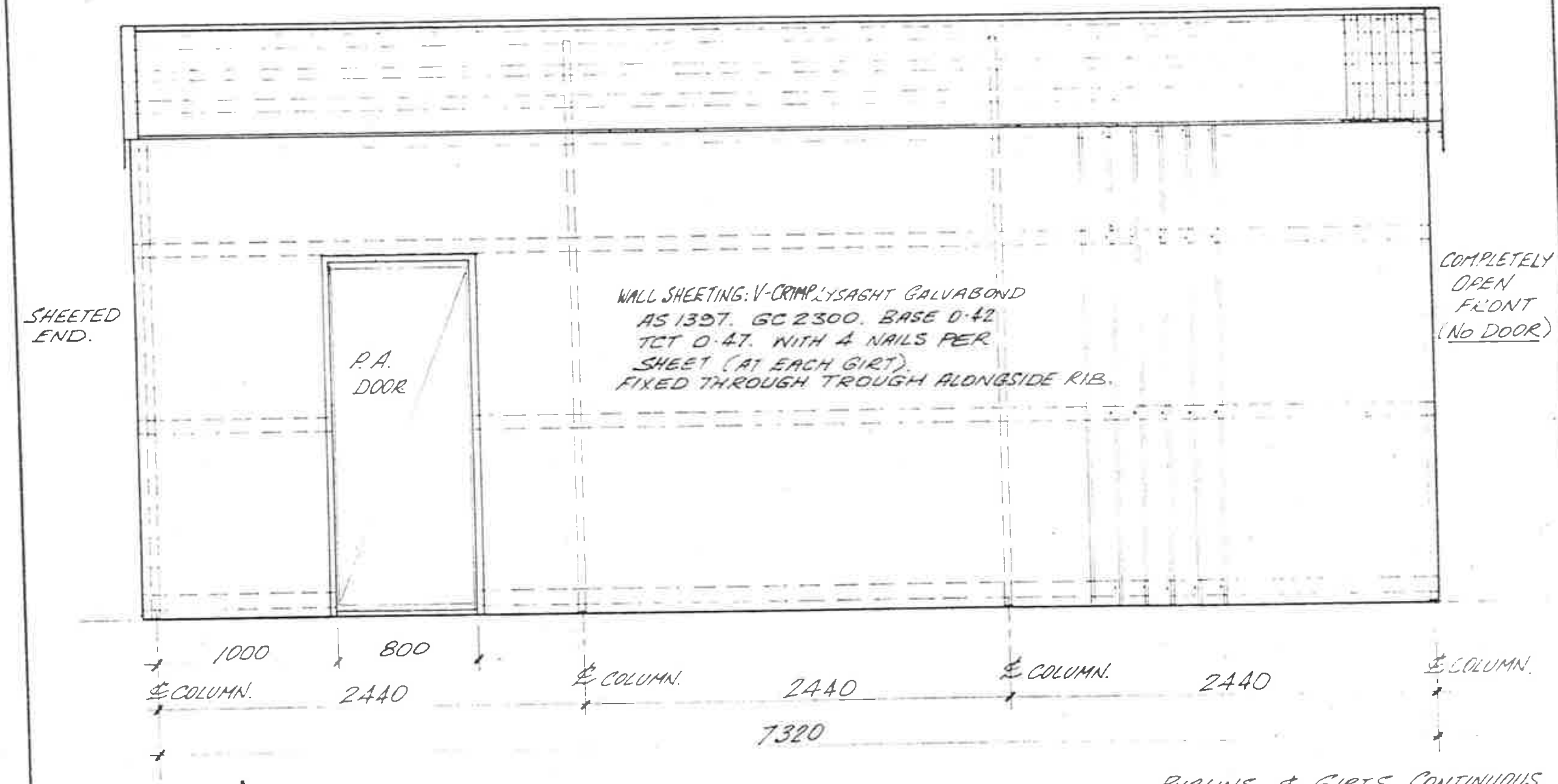
APPENDIX E - DRAWINGS

The following drawings numbers DG1 - 8 drawn by D. J. Adams, consultant draftsman to P. Wyten and Sons, and checked by the author show the arrangement and details of the tested garage.

A

REFER DRG No. DG-2
FOR SECTION A-A DETAIL.

ROOFING: CUSTOM ORB (L.B.I.)
WITH 3- ROOFING NAILS PER SHEET
WITH 1- NAIL AT LAP (AT EACH PURLIN)
FIXED THROUGH CREST.

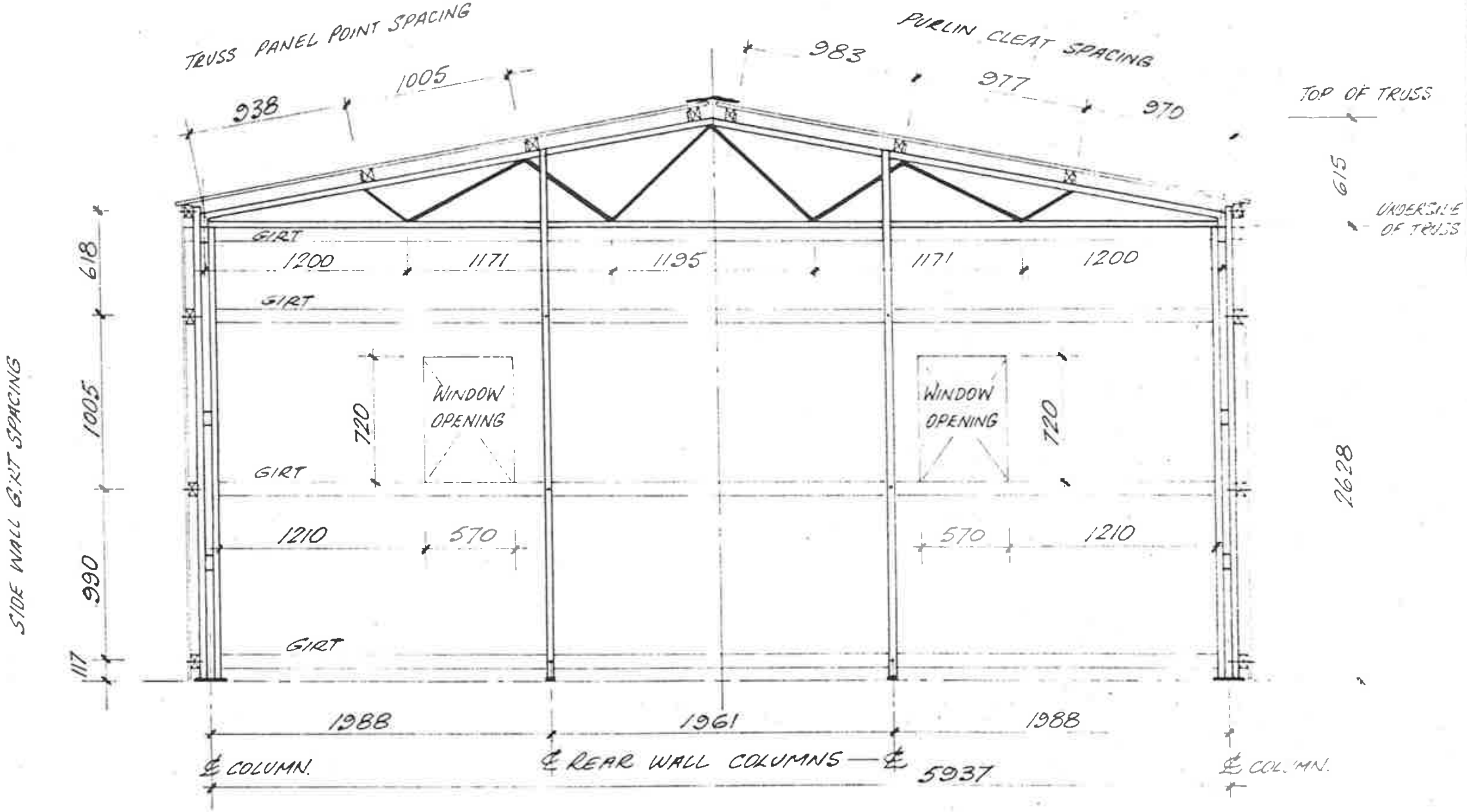


A

SIDE ELEVATION

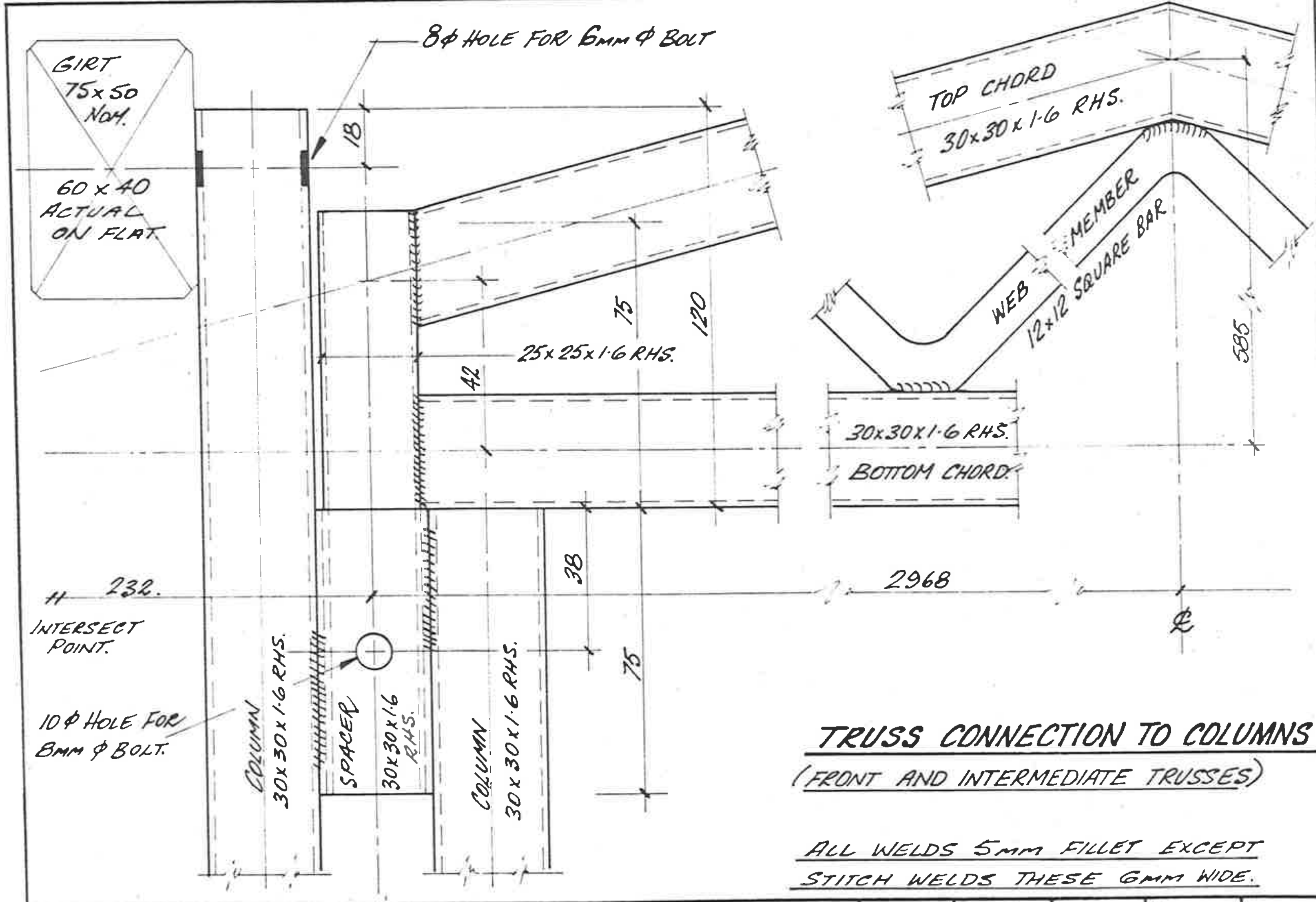
PURLINS & GIRTS CONTINUOUS
MEMBERS NOM 75 x 50 KAPUR
ACTUAL 60 x 40. PURLINS ON EDGE
GIRTS ON FLAT.

P. WYTEN & SON Pty. Ltd. HOOKINA. St. SALISBURY Nth.	STRUCTUAL DETAILS OF LOAD TESTED DOMESTIC GARAGE	DRAWN	D.J. ADAMS.	SCALE	1:25	DRG No
		CH'KD	L. [illegible]	DATE	OCT 1977	DG-1.



SECTION VIEW A-A

P. WYTEN & SON Pty. Ltd. HOOKINA. St. SALISBURY Nth.	STRUCTUAL DETAILS OF LOAD TESTED DOMESTIC GARAGE	DRAWN	D.J. ADAMS	SCALE	1:25	DRG No
		CHKD	L Nash	DATE	OCT 1977	DG-2.

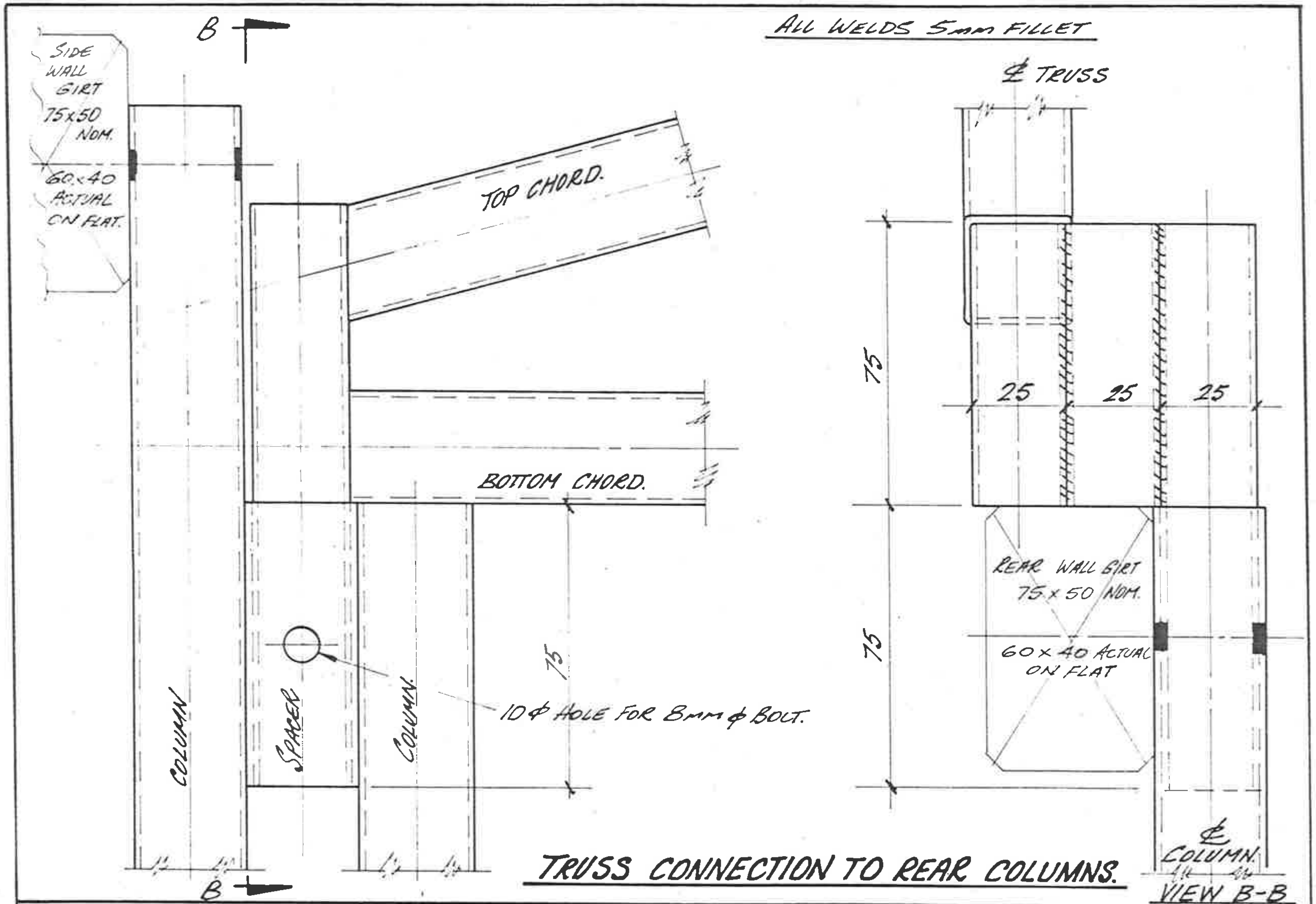


TRUSS CONNECTION TO COLUMNS

(FRONT AND INTERMEDIATE TRUSSES)

ALL WELDS 5MM FILLET EXCEPT STITCH WELDS THESE 6MM WIDE.

P. WYTEN & SON Pty. Ltd.	STRUCTUAL DETAILS OF LOAD TESTED	DRAWN	D.J. ADAMS	SCALE	FULL SIZE.	DRG No
HOOKINA. St. SALISBURY Nth.	DOMESTIC GARAGE	CH'KD	L Clark	DATE	OCT 1977	DG-3



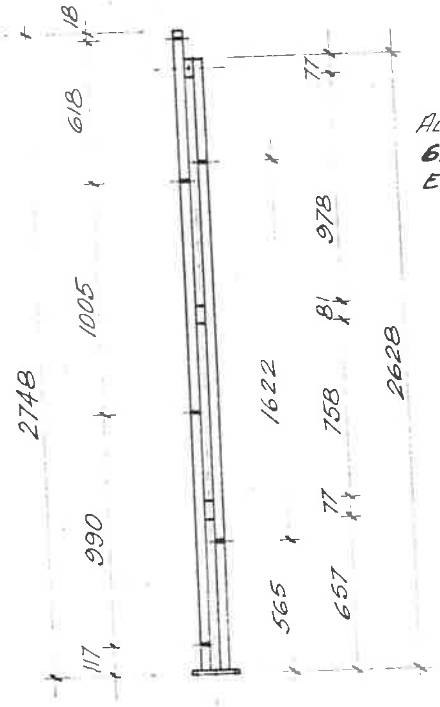
P. WYTEN & SON Pty. Ltd. HOOKINA. St. SALISBURY Nth.	STRUCTUAL DETAILS OF LOAD TESTED DOMESTIC GARAGE	DRAWN	D.J. ADAMS	SCALE	FULL SIZE.
		CH KD	L. Clark	DATE	OCT 1977

DRG No DG-4

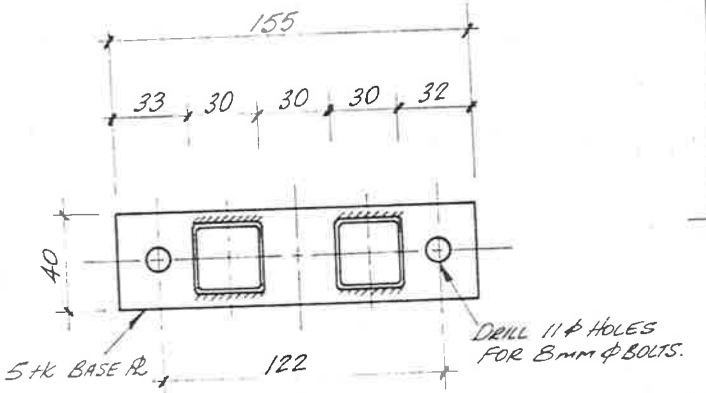
ALL WELDS 5MM FILLET.

ALL TUBES & SPACERS
30×30×1·6 R.H.S.

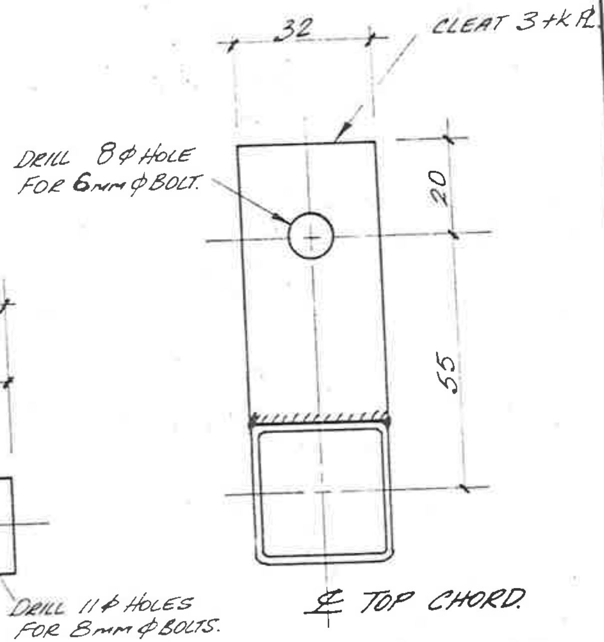
ALL HOLES 8Φ FOR
6MM Φ BOLTS.
EXCEPT WHERE STATED.



COLUMN DETAIL



COLUMN BASE PLATE
DETAIL



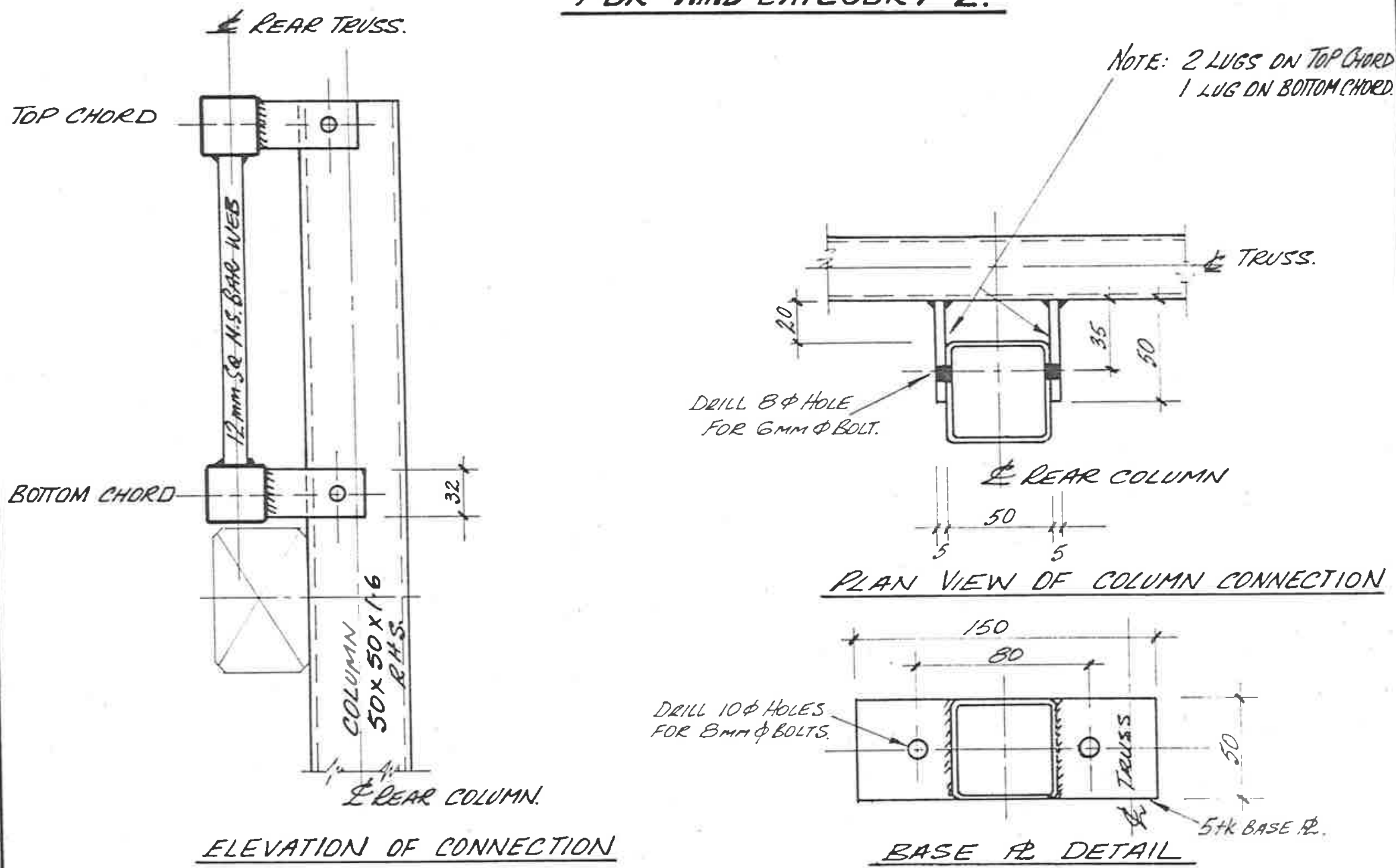
PURLIN CLEAT
DETAIL

P. WYTEN & SON Pty. Ltd. HOOKINA. St. SALISBURY Nth.	STRUCTUAL DETAILS OF LOAD TESTED DOMESTIC GARAGE	DRAWN	D.J. ADAMS	SCALE	1:20, 1:2 & FULL SIZE.	DRG No
		CH'KD	L. Clark	DATE	OCT 1977	DG-5

REAR WALL COLUMN DETAIL

ALL WELDS 5mm FILLET.

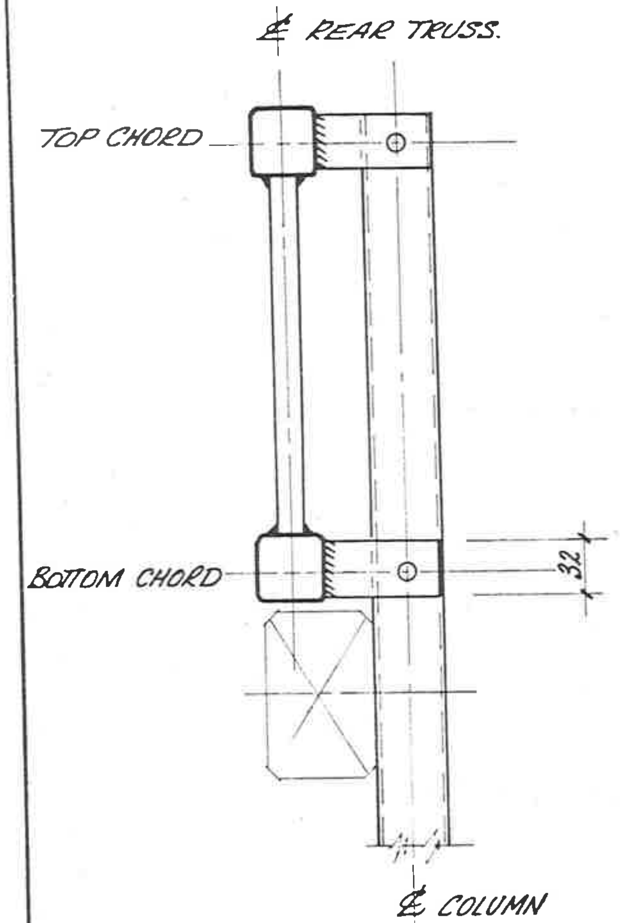
FOR WIND CATEGORY 2.



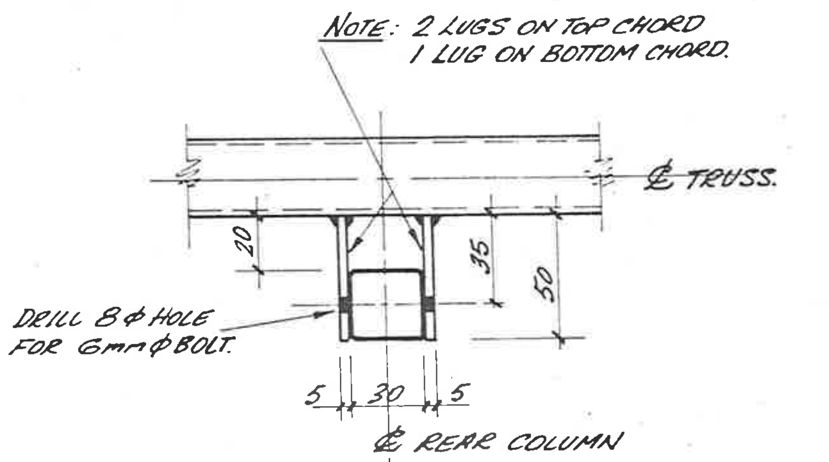
P. WYTEN & SON Pty. Ltd.	STRUCTUAL DETAILS OF LOAD TESTED	DRAWN	D.J. ADAMS	SCALE	1:2	DRG No
HOOKINA. St. SALISBURY Nth.	DOMESTIC GARAGE	CH'KD	L. Nash	DATE	OCT 1977	DG-6

REAR WALL COLUMN DETAIL FOR WIND CATEGORY 3.

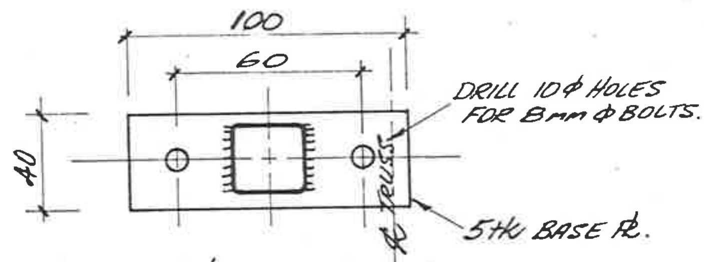
ALL WELDS 5mm FILLET.



ELEVATION OF CONNECTION

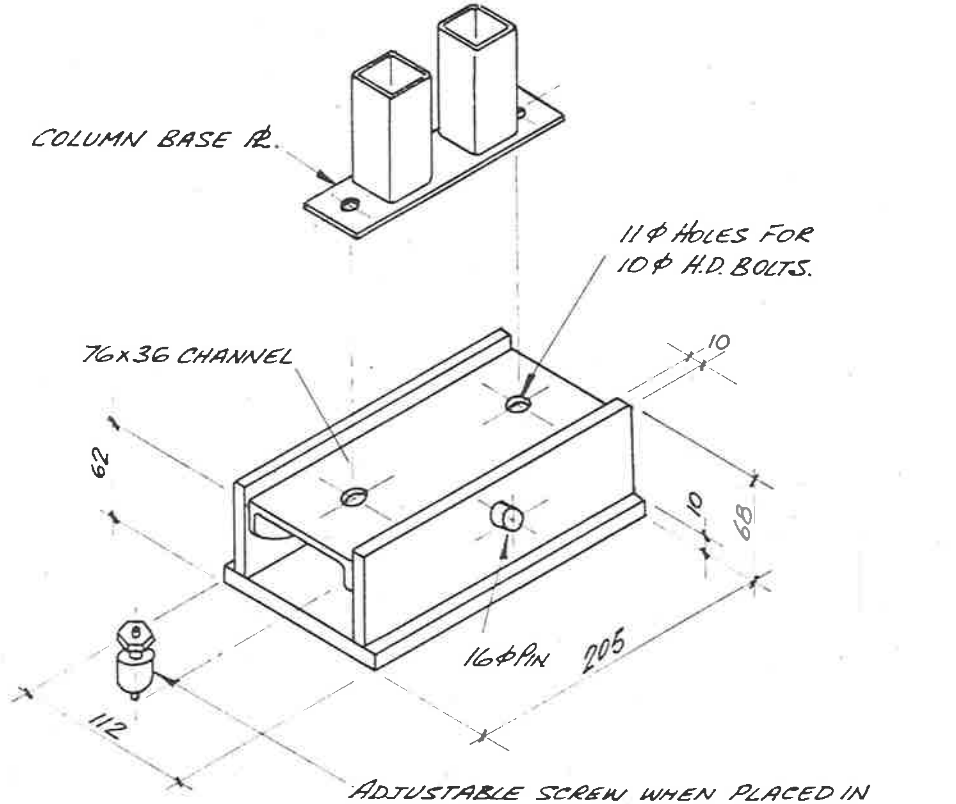


PLAN VIEW OF CONNECTION



BASE PL DETAIL

P. WYTEN & SON Pty. Ltd.	STRUCTUAL DETAILS OF LOAD TESTED	DRAWN	D.J. ADAMS	SCALE	1:2	DRG No
HOOKINA St. SALISBURY Nth.	DOMESTIC GARAGE	CH'KD	L. Nash	DATE	OCT 1977	DG-7



ISOMETRIC VIEW TEST RIG FOR BASE PLATE GARAGE COLUMNS.

P. WYTEN & SON Pty. Ltd. HOOKINA. St. SALISBURY Nth.	STRUCTUAL DETAILS OF LOAD TESTED DOMESTIC GARAGE	DRAWN	P.J. ADAMS.	SCALE	N.T.S.	DRG No
		CH'KD	L Clark	DATE	OCT 1977.	DG-8

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