STRUCTURAL HANDBOOK
MARCH 2015

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THE LOCKWOOD SYSTEM
STRUCTURAL HANDBOOK MARCH 2015

INDEX

SECTION F PANEL PRESSURE TESTS

THE LOCKWOOD SYSTEM STRUCTURAL HANDBOOK

SECTION A GENERAL

1. DESCRIPTION OF THE LOCKWOOD STRUCTURAL SYSTEM

The Lockwood System utilizes solid timber Pinus Radiata wall planks and roofing sarking each tongued and grooved to interlock with the adjacent planks. Wall panels are stiffened by the intersection of either return walls, or laminated timber stiffening members. All returns or stiffening elements are rigidly connected by means of extruded aluminium "profiles" which are tight fitting and form an effective "dovetail" between the adjacent elements.

Roof sarking is stiffened where necessary by laminated timber roof beams spaced to adequately restrict stresses and deflections in the sarking system.

2. FORMAT OF HANDBOOK

A Design Section, Section B, is included in this handbook to assist in the design of Lockwood residential structures in New Zealand.

Calculations are provided to justify the critical structural aspects of the Lockwood method of construction. Design criteria are established, and test information is provided. Each section is indexed in letter and number form - e.g. Page A-1. This provides the facility to be able to introduce updated information as required by code or product changes, the availability of additional test data etc.

Any future additions or alterations to the handbook will be lettered, numbered and dated. With any reissue of amended or additional pages to the handbook, a complete full index of contents will be provided. This will enable check to be made at any stage, that the handbook is complete and up to date.

3. NEW ZEALAND STANDARDS

The following New Zealand Standards have been used in the preparation of this handbook :

AS/NZS1170:2002 “Structural Design Actions”, NZS3603:1993 (including Amendment No. 4), “Timber Structures Standard”, NZS3604:2011, “Timber Framed Buildings”, NZS3606:1987, “Specification for the Manufacture of Glue Laminated Timber”, and AS/NZS1328:1998, “Glue Laminated Structural Timber”

4. COPYRIGHT

5. BSK CONSULTING ENGINEERS LTD

BSK Consulting Engineers are a Rotorua based firm of Consulting Civil and Structural Engineers. The practice maintains a close association with the timber industry and have been principal consultants to Lockwood Buildings Limited since 1968.

SECTION B DESIGN OF LOCKWOOD STRUCTURES

This section has been produced to assist in the design of Lockwood residential structures for New Zealand Wind and Earthquake conditions.

CODE COMPLIANCE

All data in this section of the Handbook has been based on design and testing carried out in strict accordance with the Verification Method B1/VM1 of the New Zealand Building Code Handbook.

Providing this Design section of the Handbook is used correctly, resulting designs will comply with the requirements of Clause B1 Structure of the New Zealand Building Code.

SUB-INDEX

1. EXTERIOR WALL PANEL SIZES - 62mm LOCKWOOD EXTERIOR WALL SYSTEM

Tests have substantiated the following maximum panel sizes, incorporating window joinery.

(i)	N2 Window Panel	4.656m O/A	Page F2
(ii)	N3 Window Panel	4.800m O/A	Page F3
(iii)	PS Window Panel	3.876m O/A	Page F4

These pressure tests were carried out in 1991. In 1995 Lockwoods changed to the Nalco Nulook joinery system. The outer frames of all windows in the Nulook system are slightly stiffer (and stronger) than the joinery used in the tests. The test results are therefore still valid.

The Nalco Nulook ranch slider outer frame is however considerably weaker than the previous Lockwood joinery. Previously advantage was taken of the stiffness of the ranch-slider jamb sections to increase exterior wall panel lengths above the 4.5m maximum where a ranch slider was incorporated in an exterior wall. This no longer applies with the weaker Nulook ranch slider joinery, and panels should be restricted to 4.5m or additional laminated wall stiffeners incorporated into the design.

It is Lockwood Standard practice to limit all 62mm Lockwood wall panel lengths to 4.5m maximum where not covered by the above mentioned tests.
Should room dimensions require exterior wall panel lengths longer than can be provided within the above criteria, additional wall stiffening elements should be introduced to provide additional lateral restraints (return walls or laminated stiffener posts).

Refer B4 for design of laminated columns.

2. EXTERIOR WALL PANEL SIZES - 44mm WALL BATTENED SYSTEM WITH CONVENTIONAL CLADDING OR BRICK VENEER

Tests have substantiated the following maximum panel sizes, incorporating window joinery.

(i) IH2 window panel - 3.244m O/A. 46 x 46 wall battens at 600mm c/c. Minimum width of solid panel adjacent to the window opening of 770mm.

(ii) IH2 window panel - 3.656m O/A. Standard 46 x 46 wall battens at 600mm c/c plus two additional full height 46 x 46 wall battens on each side of the window.

Should room dimensions require exterior wall panel lengths longer than can be provided within the above criteria, additional wall stiffeners should be introduced to provide additional lateral restraints.

Refer B4 for design of laminated columns.

3. EXTERIOR WALL LAMINATED STIFFENER POSTS

(i) EXTERIOR WALL STIFFENER POST DESIGN

As mentioned in Section A, Clause 1, Lockwood wall panels are stiffened by the intersection of either return walls, or laminated timber stiffener columns.

The following design charts (refer pages B5-B8) are given to determine stiffener post sizes required for given Wind Zones, stiffener lengths and lengths of exterior wall stabilised by the column (loaded dimension).

These design charts assume that the laminated column is X-profiled full height to a section of solid wall, and that for columns supporting roof beams (e.g. centrally on gable end walls), at least one 10mmø tie rod is located adjacent to the column.

Structural design of these stiffener columns has been based on structural grade GL8 Glulam columns in terms of AS/NZS 1328.1-1998 and using the “Timber Structures Standard”, NZS3603:1993.

(ii) EXTERIOR WALL LAMINATED STIFFENER POSTS – TOP & BOTTOM FIXINGS

All exterior wall laminated stiffener posts are to be effectively fixed top and bottom as below.

(a) At Floor Level One of the Following Fixings Shall be Provided:-

(i) Provide two Pryda Multigrips nail fixed to the sides of each stiffener and to the timber floor or floor plate, in accordance with the Lockwood Standard Details.

or

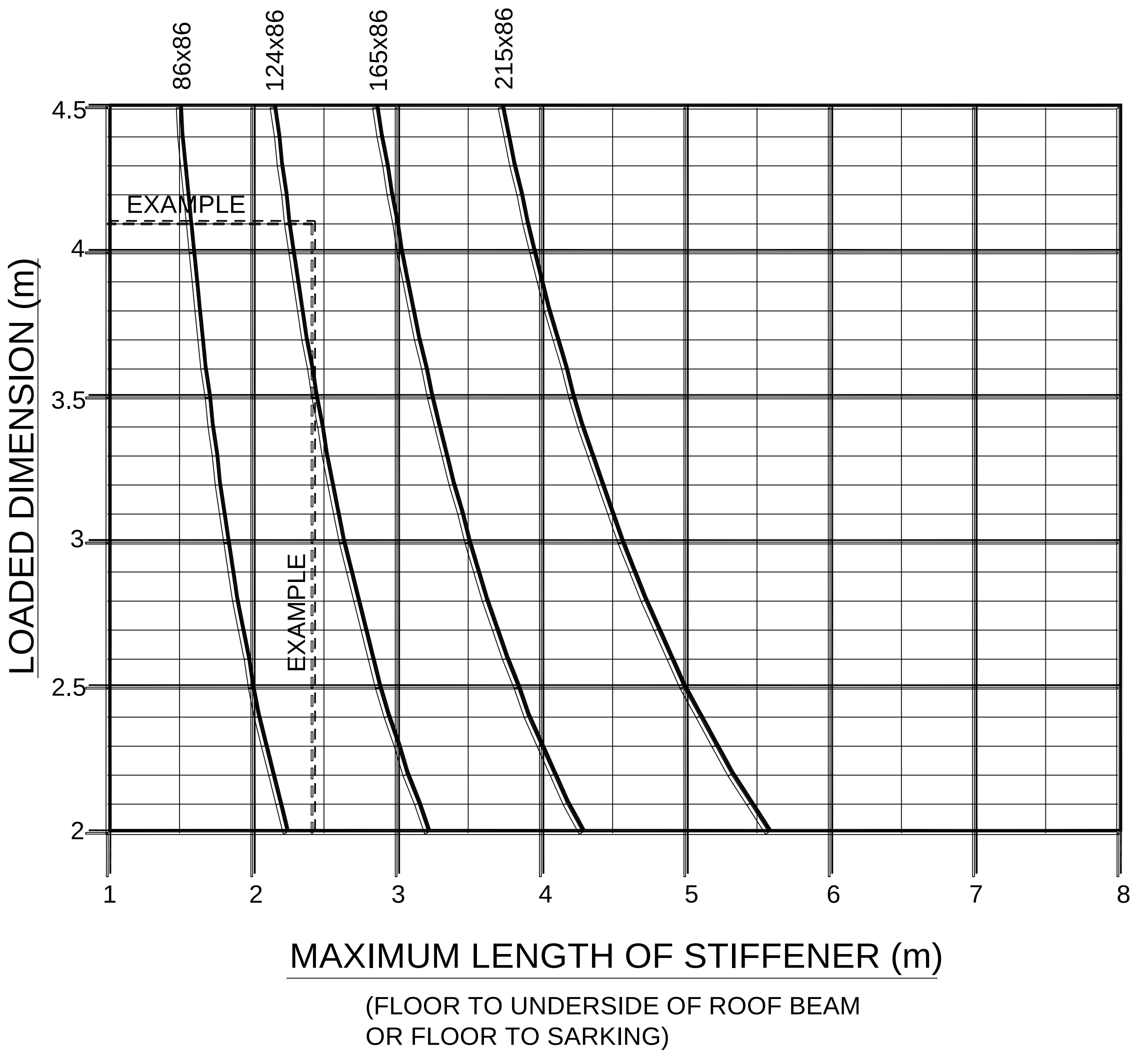
(ii) Carry stiffener through floor and bolt fix to subfloor timbers in accordance with Lockwood Standard Detail B18.

(b) At Roof Level Provide Fixings as Below:-

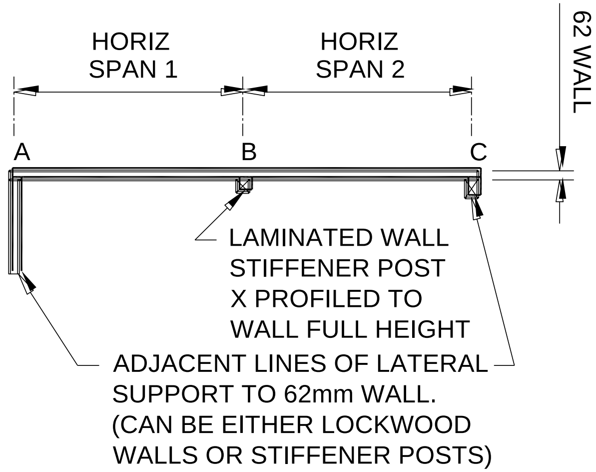
(i) Stiffeners at side walls – extend stiffener through roof sarking and cut sarking neatly right around stiffener.

(ii) Stiffeners at end walls supporting beams – tenon and mortice in accordance with Lockwood Standard Details (M12 galvanised pins).

LOCKWOOD STIFFENER SELECTION CHART
VERY HIGH WIND ZONE (TO NZS 3604:1999)



DEFINITION OF LOADED DIMENSION



$$\text{LOADED DIMENSION} = \frac{(\text{HORIZ SPAN 1} + \text{HORIZ SPAN 2})}{2}$$

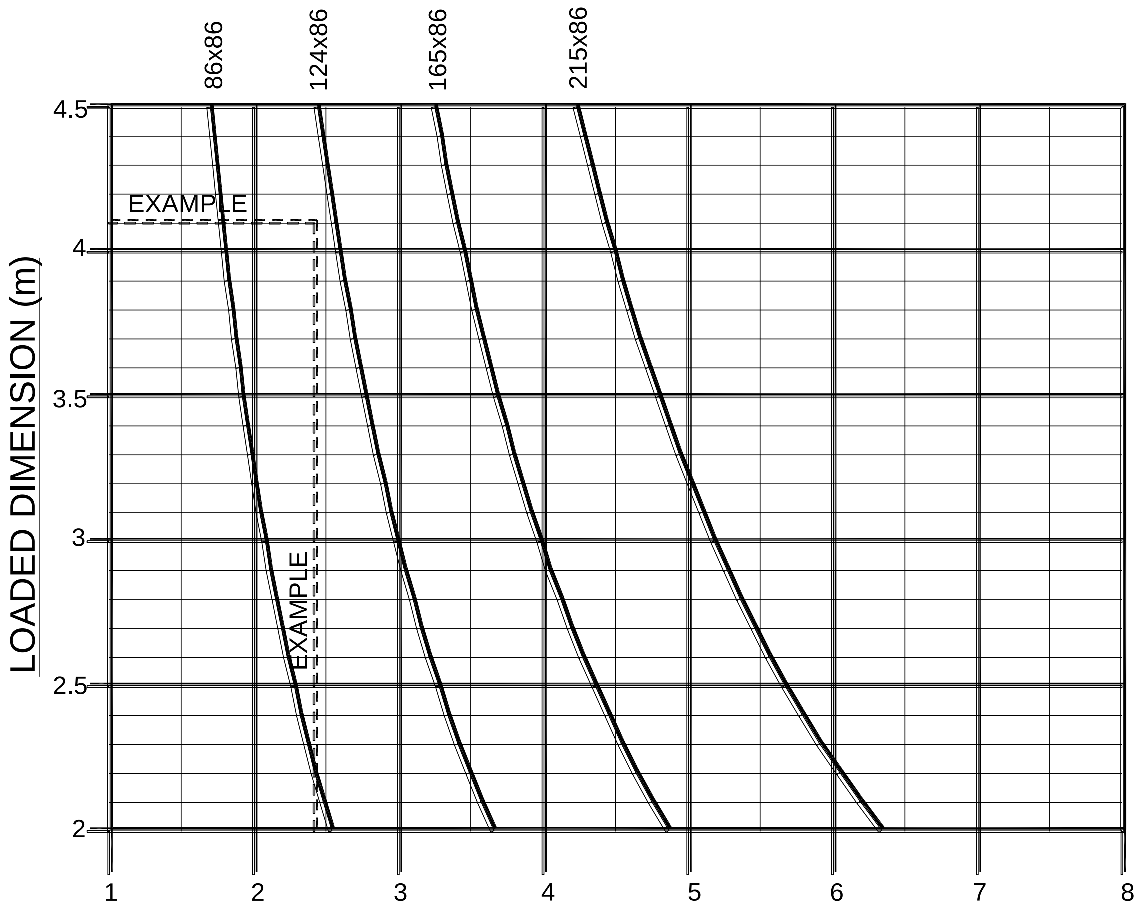
N.B. WALL PANELS BETWEEN LINES OF LATERAL SUPPORT A, B AND C MAY CONTAIN SINGLE JOINERY UNITS (REFER PAGE B-2 OF HANDBOOK FOR MAXIMUM PANEL SIZES)

EXAMPLE

A POST 2418mm HIGH (14 BOARD STUD) WITH A LOADED DIMENSION OF 4.1m REQUIRES A 165x86 LAM STIFFENER POST

VERY HIGH WIND ZONE

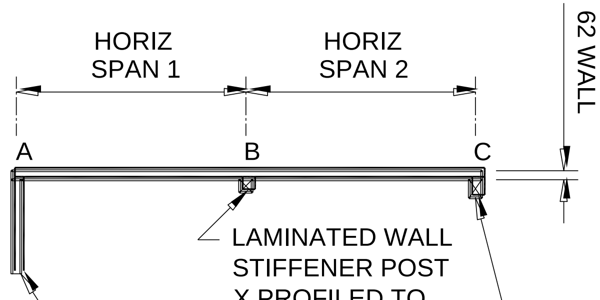
LOCKWOOD STIFFENER SELECTION CHART
HIGH WIND ZONE (TO NZS 3604:1999)



MAXIMUM LENGTH OF STIFFENER (m)

(FLOOR TO UNDERSIDE OF ROOF BEAM
OR FLOOR TO SARKING)

DEFINITION OF LOADED DIMENSION



LAMINATED WALL STIFFENER POST X PROFILED TO WALL FULL HEIGHT

ADJACENT LINES OF LATERAL SUPPORT TO 62mm WALL. (CAN BE EITHER LOCKWOOD WALLS OR STIFFENER POSTS)

$$\text{LOADED DIMENSION} = \frac{(\text{HORIZ SPAN 1} + \text{HORIZ SPAN 2})}{2}$$

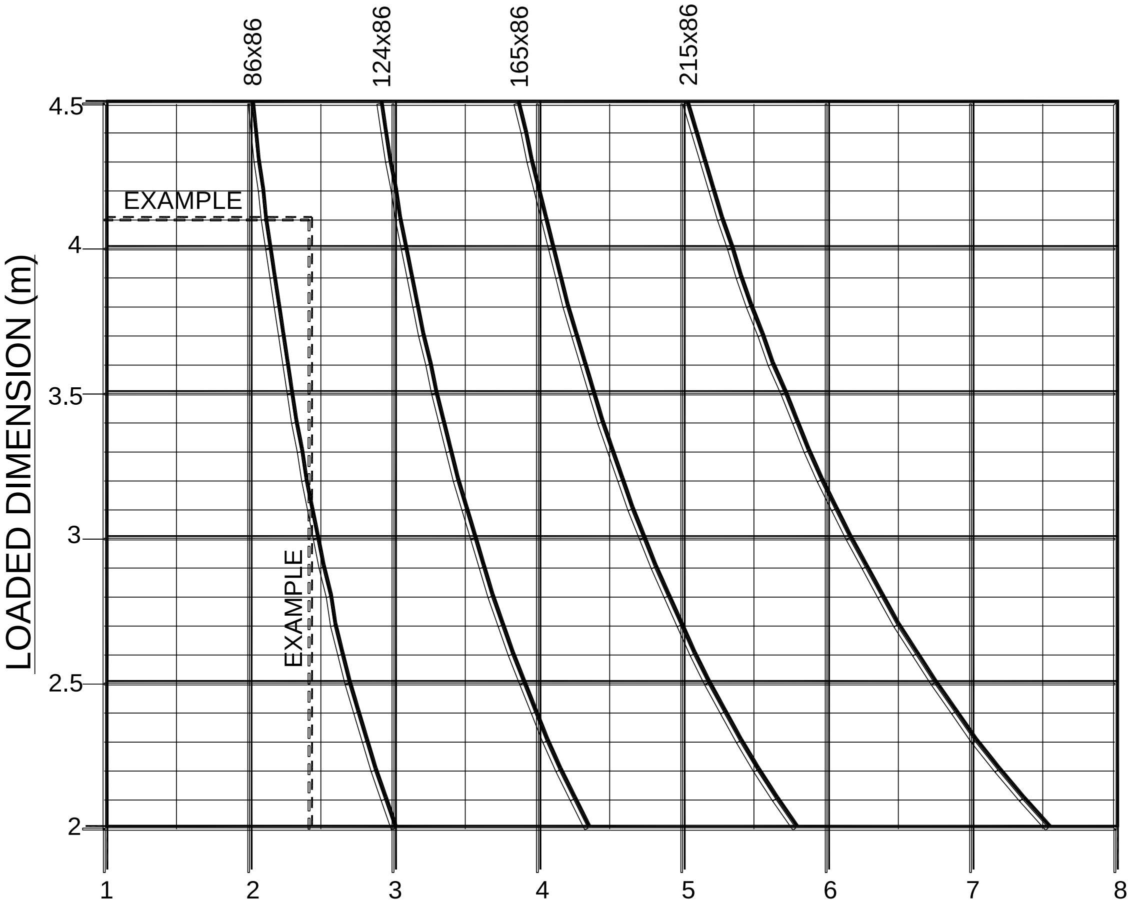
N.B. WALL PANELS BETWEEN LINES OF LATERAL SUPPORT A, B AND C MAY CONTAIN SINGLE JOINERY UNITS (REFER PAGE B-2 OF HANDBOOK FOR MAXIMUM PANEL SIZES)

EXAMPLE

A POST 2418mm HIGH (14 BOARD STUD) WITH A LOADED DIMENSION OF 4.1m REQUIRES A 124x86 LAM STIFFENER POST

HIGH WIND ZONE

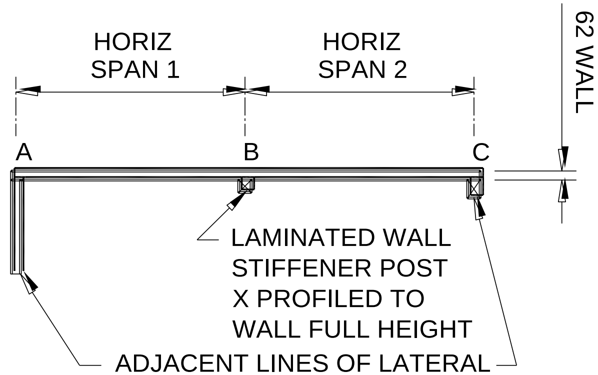
LOCKWOOD STIFFENER SELECTION CHART
MEDIUM WIND ZONE (TO NZS 3604:1999)



MAXIMUM LENGTH OF STIFFENER (m)

(FLOOR TO UNDERSIDE OF ROOF BEAM
OR FLOOR TO SARKING)

DEFINITION OF LOADED DIMENSION



$$\text{LOADED DIMENSION} = \frac{(\text{HORIZ SPAN 1} + \text{HORIZ SPAN 2})}{2}$$

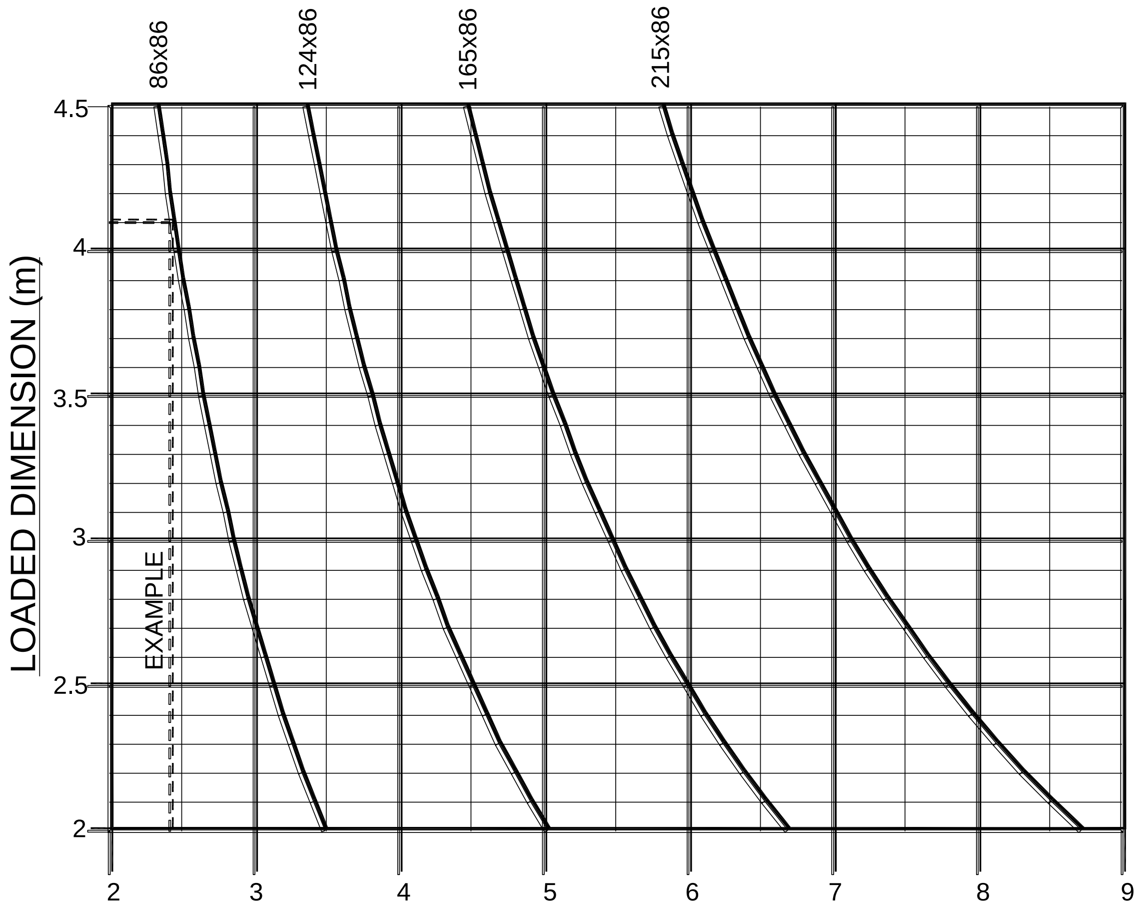
N.B. WALL PANELS BETWEEN LINES OF LATERAL SUPPORT A, B AND C MAY CONTAIN SINGLE JOINERY UNITS (REFER PAGE B-2 OF HANDBOOK FOR MAXIMUM PANEL SIZES)

EXAMPLE

A POST 2418mm HIGH (14 BOARD STUD) WITH A LOADED DIMENSION OF 4.1m REQUIRES A 124x86 LAM STIFFENER POST

MEDIUM WIND ZONE

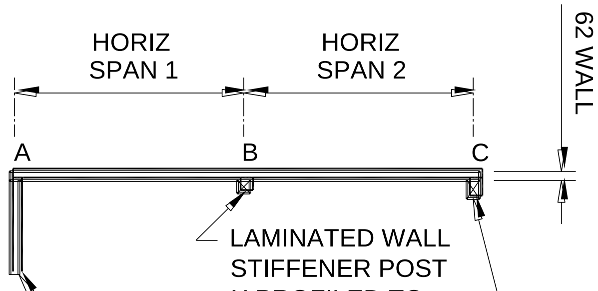
LOCKWOOD STIFFENER SELECTION CHART
LOW WIND ZONE (TO NZS 3604:1999)



MAXIMUM LENGTH OF STIFFENER (m)

(FLOOR TO UNDERSIDE OF ROOF BEAM
OR FLOOR TO SARKING)

DEFINITION OF LOADED DIMENSION



LAMINATED WALL STIFFENER POST X PROFILED TO WALL FULL HEIGHT

ADJACENT LINES OF LATERAL SUPPORT TO 62mm WALL. (CAN BE EITHER LOCKWOOD WALLS OR STIFFENER POSTS)

$$\text{LOADED DIMENSION} = \frac{(\text{HORIZ SPAN 1} + \text{HORIZ SPAN 2})}{2}$$

N.B. WALL PANELS BETWEEN LINES OF LATERAL SUPPORT A, B AND C MAY CONTAIN SINGLE JOINERY UNITS (REFER PAGE B-2 OF HANDBOOK FOR MAXIMUM PANEL SIZES)

EXAMPLE

A POST 2418mm HIGH (14 BOARD STUD) WITH A LOADED DIMENSION OF 4.1m REQUIRES A 86x86 LAM STIFFENER POST

LOW WIND ZONE

4. LATERAL STABILITY - BRACING UNIT APPROACH

THE LOCKWOOD SYSTEM
STRUCTURAL HANDBOOK 2006
ADDENDUM 1 (OCTOBER 2013)

I. INTRODUCTION

This document amends the Lockwood Structural Handbook, 2006.

The scope of this document is limited to the use of internal 44mm walls and external 107mm walls for the provision of lateral bracing requirements of the New Zealand Building Code, Clause B1 Structure.

The results presented herein have been derived from experimental testing completed by Holmes Solutions in accordance with the BRANZ EM3-V3 testing protocol with the displacement targets modified to reflect the requirements of AS / NZS 1170.1 for non-plaster lined walls. At the completion of the testing, calculations of the Bracing units (BU) capacity of the tested walls were made in accordance to EM3-V3 analyses, with revised F1 and F2 factors to reflect the manufactured form of the walls.

The new Lockwood 107mm insulated external wall system has been verified by BSK Consulting Engineers Ltd to have structural performance at least as good as the 62mm board system referenced in the Lockwood Structural Handbook, 2006.

Lateral bracing calculations completed in accordance with this Addendum, for structures designed in accordance with the Lockwood Structural Handbook, will comply with the requirements of Clause B1 Structure of the New Zealand Building code.

II. EXTERNAL REPORTS

This document makes use of the findings contained in the following external reports:

Lockwood Report 19175 (107mm wall board), BSK Consulting Engineers Limited, February 2013.

Lockwood Wall Bracing Capacity Testing - Report 109800.00.01 (v1.2), Holmes Solutions LP, September 2013

III. COPYRIGHT

IV. CHANGES TO SECTION B- DESIGN OF LOCKWOOD STRUCTURES

B.4. LATERAL STABILITY - BRACING UNIT APPROACH

This clause replaces Section B. Clause 4. LATERAL STABILITY – BRACING UNIT APPROACH. Pages B-9 to B-12, in its entirety.

(a) The lateral stability of the Lockwood form of construction relies on the stiffness that is provided by walls acting as shear panels

(b) When determining Bracing Demand, as set out in NZS 3604:2011 Clauses 5.2 and 5.3, where using roof cladding with a weight no greater than 20kg/m^2 (i.e. most iron roofs) and Lockwood exterior walls, use light roof and wall data throughout in terms of NZS3604:2011, (refer Page X-8 for verification of roof and wall weights).

(c) Bracing Capacity is provided by sections of 44mm Internal and 107mm External walls which satisfy the following conditions:

(1) At least 1.0m in length;

(2) A wall height between 2.2m (13 planks) and 3.6m (21 planks);

*Note: Bracing capacity given in the tables below are valid for 2.2m (13 plank) and 2.4m (14 plank) high walls. Walls higher than 2.4m have reduced bracing capacity calculated by multiplying the given value by **f = 2.4 / height**.*

(3) Is a blank section of wall that contain no windows, doors or other openings;

Note: Wall sections used as bracing elements may be part of a longer wall as long as the conditions above are satisfied.

(4) Contains a minimum of two tie rods, one installed within 150mm of each end of the wall section;

(5) Contains a minimum of one aluminium profile within the length of the section;

Note: Bracing capacity given in the tables below are valid for wall sections containing two aluminium profiles. Wall sections with only 1 profile have reduced bracing capacity calculated by multiplying the given value by 0.8.

Tables B.4.1, B.4.2, B.4.3 and B.4.4 below give Bracing Unit Capacities for wall sections meeting the conditions above, for walls containing two or more aluminium profiles.

Table B.4.1 – Bracing capacity table for 44mm internal wall sections, no openings, 2 tie-rods, 2 aluminium profiles

Length [m]	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**	**2.2**	**2.4**	**2.6**	**2.8**	**3.0**
EQ [BU]	*148	*151	154	158	161	165	168	171	175	178	182
Wind	98	102	106	110	113	117	121	125	129	133	136

[BU]											
Length [m]	**3.2**	**3.4**	**3.6**	**3.8**	**4.0**	**4.2**	**4.4**	**4.6**	**4.8**	**5.0**	**over 5.0**
EQ [BU]	185	188	192	195	199	202	206	209	212	216	216
Wind [BU]	140	144.5	148	152	155	159	163	167	171	175	175

**Note: If placed on a timber-framed floor these values must be reduced to 120 B/U per metre.*

Table B.4.2 – Bracing capacity table for External 107mm wall sections, no openings, 2 tie-rods, 2 aluminium profiles

Length [m]	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**	**2.2**	**2.4**	**2.6**	**2.8**	**3.0**
EQ [BU]	*150	*171	*174	176	179	181	183	186	188	191	193
Wind [BU]	106	111	116	120	125	129	134	138	143	147	152
Length [m]	**3.2**	**3.4**	**3.6**	**3.8**	**4.0**	**4.2**	**4.4**	**4.6**	**4.8**	**5.0**	**over 5.0**
EQ [BU]	196	198	201	203	206	208	210	213	215	218	218
Wind [BU]	157	161	166	170	175	179	184	188	193	198	198

**Note: If placed on a timber-framed floor these values must be reduced to 120 B/U per metre.*

(d) Further information on the lateral stability of Lockwood structures is given in Section C, including testing information and an example Bracing Unit calculations for a typical house design.

B.9. TIE RODS

Section B. Clause 9. TIE RODS. Page B-25. The following text replaces item 3. of sub-part titled "Tie rods shall be provided in the following positions".

3. For each section of wall used in bracing unit calculations, one tie rod shall be installed at each end of the section length, with each tie rod being not more than 150mm from the end of the wall section.

5. FLOOR BEAMS

Design charts are provided for open floor loadings (pages B14 & B15) for both GL8 and GL10 structural grade glulam floor beams. Point loadings and line loadings should be the subject of specific beam designs.

The loadings assumed for design are as follows.

- Basic Live Load Q = 1.5 kPa
- Particle Board floor on joists and 10mm Gib Board ceiling, Dead Load G = 0.4 kPa (represented by solid lines on Design Chart)
- Lockwood 44mm floor planks, Dead Load G = 0.264 kPa (represented by dashed lines on Design Chart)

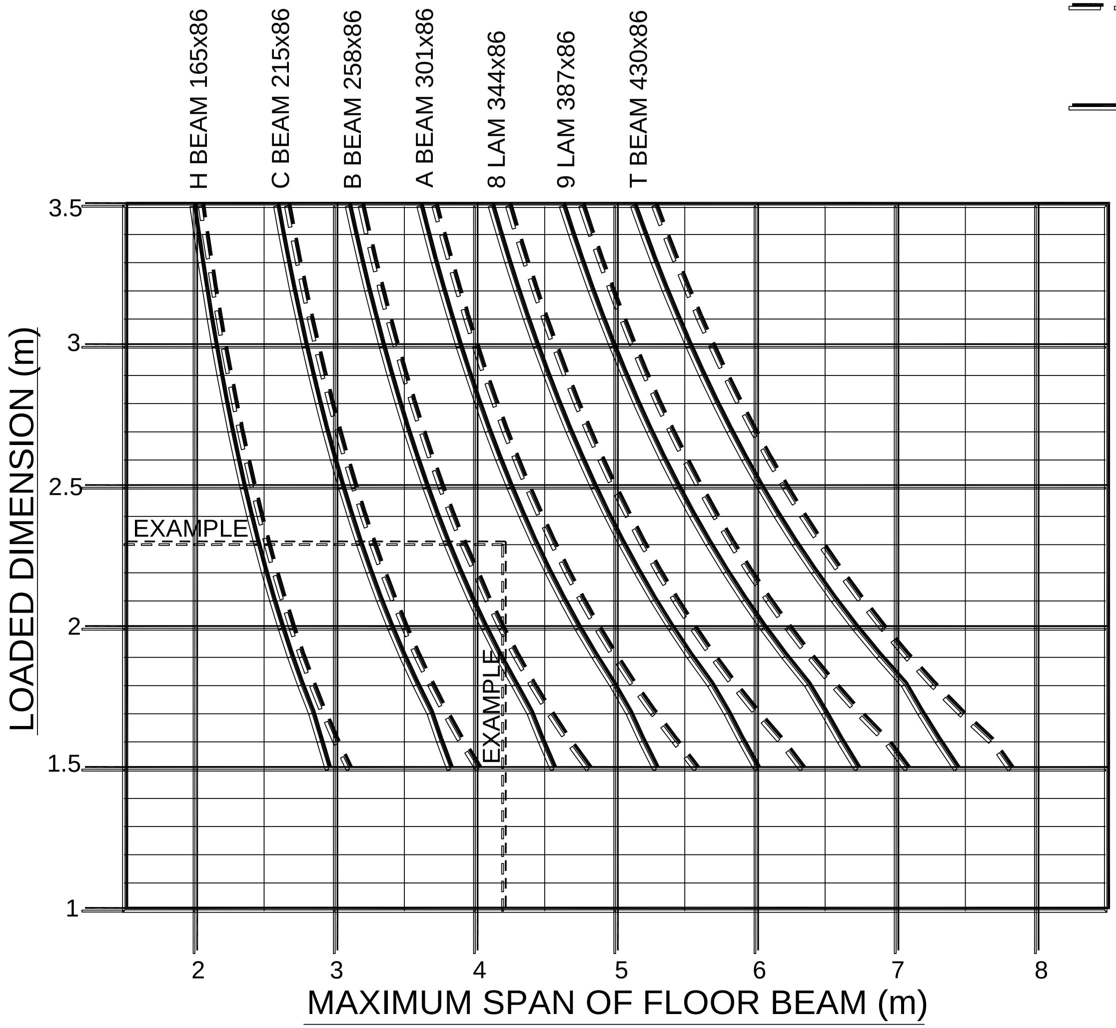
Structural design of floor beams has been based on structural grade GL8 (page B14) and GL10 (page B15) glulam beams in terms of AS/NZS 1328.1-1998 and using the "Timber Structures Standard", NZS 3603:1993.

Deflection of the floor beams given in the following chart has been limited in line with the criteria recommended for floor beams by AS/NZS 1170.0:2002 "Structural Design Actions, Part O: General Principles". Under this code the long term serviceability limit for mid-span deflection is Span/400.

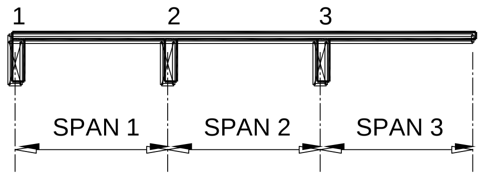
LOCKWOOD FLOOR BEAM SELECTION CHART - GL8 STRUCTURAL GRADE BEAMS

DASHED LINES REPRESENT LOCKWOOD 44mm FLOOR PLANK LOADS (APPROX 0.264 kPa DEAD LOAD)

SOLID LINES REPRESENT PARTICLE BOARD FLOOR ON JOISTS + CEILING LOADS (APPROX 0.4 kPa MAX DEAD LOAD)



DEFINITION OF LOADED DIMENSION



FOR BEAM 1 - LOADED DIM = $\frac{\text{SPAN 1}}{2}$

FOR BEAM 2 - LOADED DIM = $\frac{(\text{SPAN 1} + \text{SPAN 2})}{2}$

FOR BEAM 3 - LOADED DIM = $\left(\frac{\text{SPAN 2}}{2}\right) + \text{SPAN 3}$

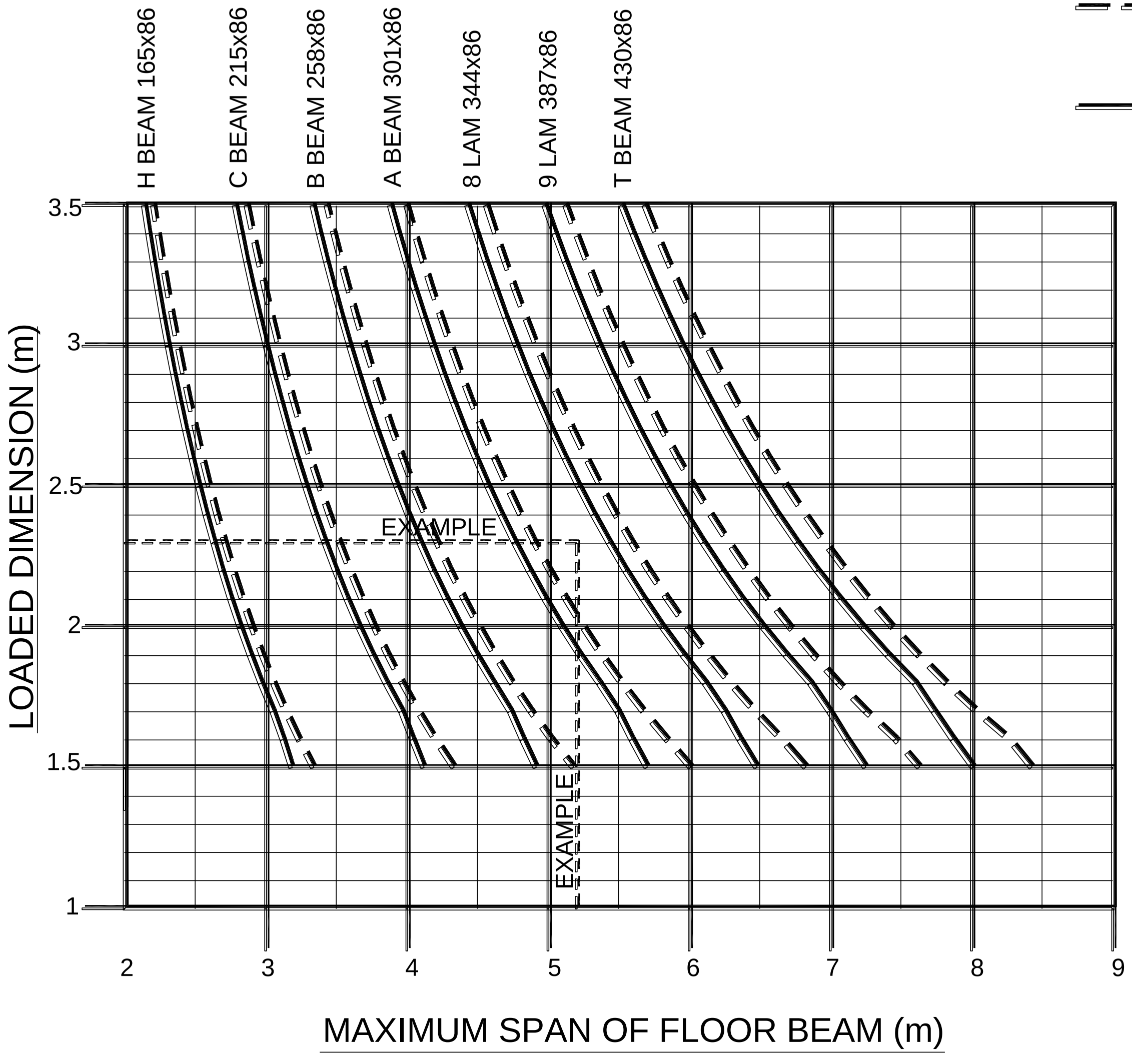
EXAMPLE

A FLOOR BEAM SPANNING 4.2m SUPPORTING 44mm LOCKWOOD FLOOR BOARDS WITH A LOADED DIMENSION OF 2.3m REQUIRES A 301x86 LAM BEAM (A BEAM)

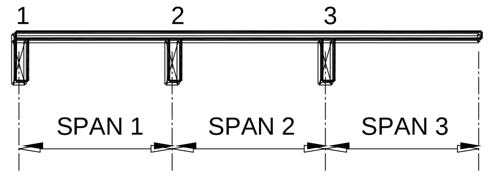
LOCKWOOD FLOOR BEAM SELECTION CHART - GL10 STRUCTURAL GRADE BEAMS

DASHED LINES REPRESENT LOCKWOOD 44mm FLOOR PLANK LOADS (APPROX 0.264 kPa DEAD LOAD)

SOLID LINES REPRESENT PARTICLE BOARD FLOOR ON JOISTS + CEILING LOADS (APPROX 0.4 kPa MAX DEAD LOAD)



DEFINITION OF LOADED DIMENSION



FOR BEAM 1 - LOADED DIM = $\frac{\text{SPAN 1}}{2}$

FOR BEAM 2 - LOADED DIM = $\frac{(\text{SPAN 1} + \text{SPAN 2})}{2}$

FOR BEAM 3 - LOADED DIM = $\left(\frac{\text{SPAN 2}}{2}\right) + \text{SPAN 3}$

EXAMPLE

A FLOOR BEAM SPANNING 5.2m SUPPORTING 44mm LOCKWOOD FLOOR BOARDS WITH A LOADED DIMENSION OF 2.3m REQUIRES A 344x86 LAM BEAM (8 LAM)

6. ROOF BEAMS

A design chart is provided for 35mm sarked roofs, for the standard Lockwood laminated beams, varying spans, spacings and roof pitches (B17).

The loadings assumed for design are 0.25 kPa Live Load and a maximum roof Dead Load of 0.392 kPa. This represents Light Roofing (0.08 kPa maximum), ex 75 x 50 purlins at 900mm c/c, 35mm Lockwood sarking, and either 90 x 45 dummy rafters at 300mm c/c or 140 x 45 dummy rafters at 450mm c/c. Where it is intended to use a Heavy Roof cladding or roof structure heavier than specified above, roof beam design should be the subject of a special design consideration.

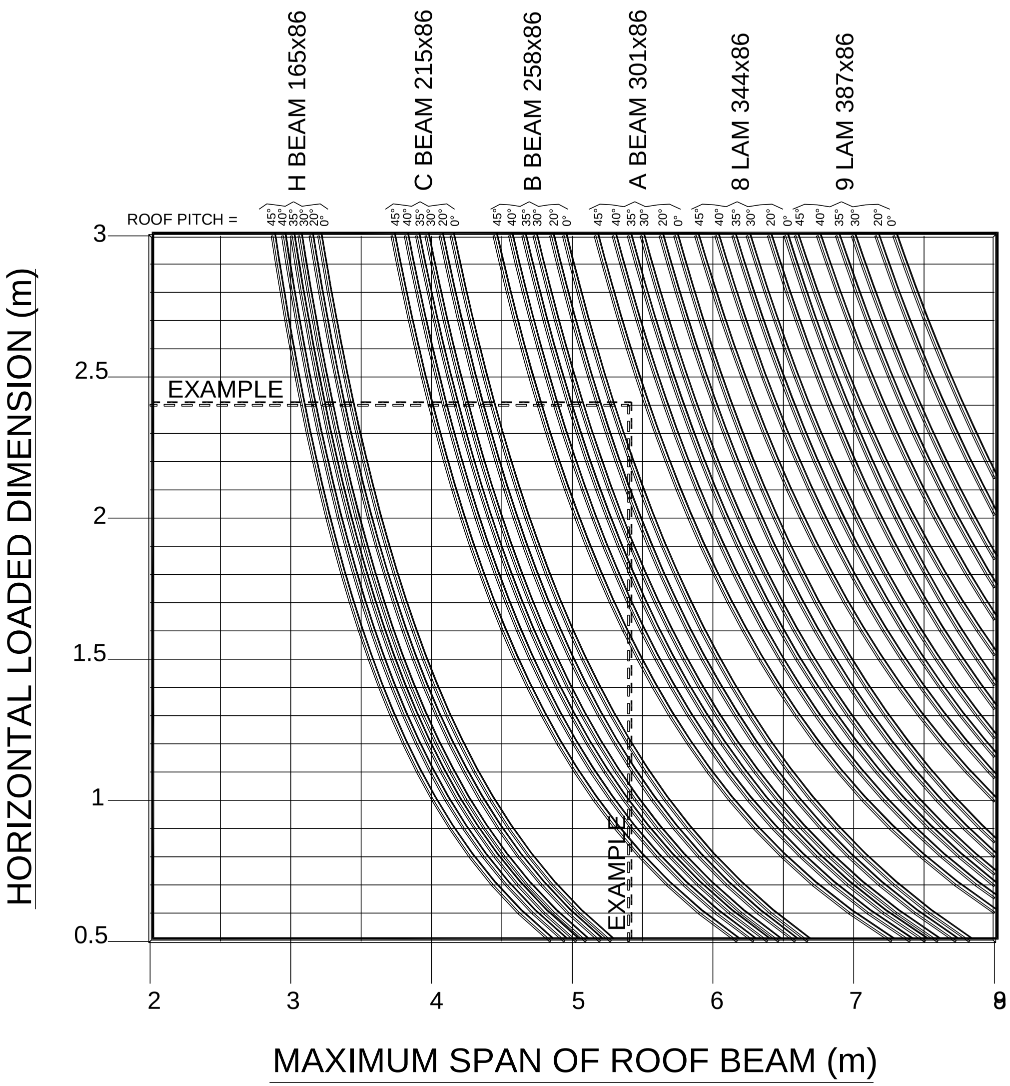
Structural design of roof beams has been based on structural grade GL8 Glulam beams in terms of AS/NZS 1328.1-1998 and using the “Timber Structures Standard”, NZS 3603:1993.

Long term deflection of the roof beams given in the following chart has been limited to the lesser of Span/320 or 25mm maximum. This is within the criteria recommended for roof members by AS/NZS 1170.0:2002, “Structural Design Actions, Part 0 : General Principles”.

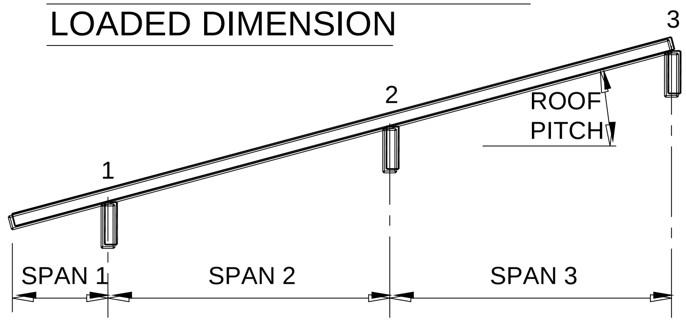
The incorporation of a concrete tile roof increases the dead load of the roof system by approximately 120%. This will result in all beam sizes increasing by at least one laminate. Sarking and lintel spans are also affected. Incorporation of a concrete tile roof should be the subject of a special design consideration.

As deflection limitation controls the design of laminated roof beams, precambering of the beams usually results in smaller member sizes. This procedure is an option for limiting beam sizes in unusually long spans for special designs.

LOCKWOOD ROOF BEAM SELECTION CHART



DEFINITION OF HORIZONTAL LOADED DIMENSION



FOR BEAM 1 - LOADED DIMENSION = SPAN 1 + $\left(\frac{\text{SPAN}}{2}\right)$

FOR BEAM 2 - LOADED DIMENSION = $\frac{(\text{SPAN 2} + \text{SPAN 3})}{2}$

FOR BEAM 3 - LOADED DIMENSION = $\frac{\text{SPAN 3}}{2}$

EXAMPLE:

A ROOF BEAM SPANNING 5.4m WITH A LOADED DIMENSION OF 2.4m REQUIRES A 301x86 LAM BEAM (A BEAM)

NOTE: THIS DESIGN CHART HAS BEEN LIMITED TO A MAXIMUM ROOF OF 8m WHICH CORRESPONDS TO A LONG TERM MIDSPAN BEAM DEFLECTION OF 25mm. ROOF BEAMS SPANNING MORE THAN 8m SHOULD BE THE SUBJECT OF A SPECIAL DESIGN

7. DUMMY RAFTERS

(i) DUMMY RAFTER DESIGN

Design charts have been provided for 35mm sarked roofs with 90 x 45 dummy rafters, and for 35mm sarked roofs with 140 x 45 dummy rafters, all dummy rafters to be VSG8 or MSG8 Radiata Pine.

The design charts provided are based on a Light Roof and Very High Wind in terms of NZS 3604:1999 (B20-B21). Where it is intended to use a Heavy Roof cladding or to build within a Specific Engineering Design Wind Area in terms of NZS 3604:1999, dummy rafters should be the subject of a special design consideration.

The maximum dummy rafter spacing is 900mm.

On site the builders must provide continuous temporary rows of support mid-span under sarking for all spans greater than 2.5m, and maintain temporary support until roof fixing is complete.

Sarking deflections should be carefully considered by the designer. Traditionally Lockwood sarked roofs have been designed to limit the long term mid-span deflection to 0.006 x span. Exceeding the 0.006 x span deflection limitation would lead to deflections which may be unacceptable – for example, if it were possible to maintain a line of sight up the roof line.

The 90 x 45 dummy rafter selection chart provided has been designed to limit long term midspan deflections to the 0.006 x span limitation.

For longer spans and where it is desired to further minimise the long term mid-span deflection, a 140 x 45 dummy rafter selection chart has also been provided. This chart has been designed to limit long term deflections to Span/300, (it is felt that for longer spans this tighter deflection limit is appropriate).

(ii) SCREW FIXING OF 90 X 45 DUMMY RAFTERS

The following dummy rafter fixing specification may be used in all wind areas under NZS3604:1999, up to and including Very High Wind, which is the 50 m/s Design Wind Speed.

Special Design wind areas under NZS3604:1999, should be the subject of special design considerations.

(a) SCREWS AND FIXING OF SCREWS

Screws are to be Type 17, 14 gauge x 115mm long, hexagonal washer face, refer Page B22.
6.3mmϕ outside threads.
Class 2 minimum durability rating, or Class 3 or Class 4.

Locate within 200mm of each end of the sarking span and at 1.000m maximum centres along the length of each dummy rafter.

Screw centrally through the 45mm width of the dummy rafter and square to the slope of the roof.

Extreme care must be exercised not to over tighten the screws, as any attempt to over tighten, or to countersink the washer could result in stripping of the timber around the screw threads and a consequential loss of holding power.

The screws shall therefore be tightened until the washer under the head is just snug onto to the top of the dummy rafter.
Care shall be taken, particularly with the first screws, to obtain an optimum torque setting to meet this requirement.

(iii) JOINTS IN 90 X 45 DUMMY RAFTERS

In most situations it is envisaged that all 90 x 45 dummy rafters will be single full length members.

Joints are permissible however near the sarking support lines and the following rules may be applied to help minimise the offcut wastage with respect to positioning of joints in 90 x 45 dummy rafters, refer Page B23):

(a) On any sarking span exceeding 2.300m horizontal span, there shall be no joints in any dummy rafter over the central 70% of the sarking span.
On sarking spans less than 2.300mm, dummy rafters may be joined at any position.

(b) At the end section, (15% of the span at each end), joints shall occur in no more than 50% of the dummy rafters, and joints shall be staggered as much as possible.

On Page B23 we have indicated the central span areas where no joints are permitted in dummy rafters, and the end of sections where joints are permitted, for typical single span and multispan situations.

Each section of each dummy rafter shall be screw fixed as specified in Clause 7(ii) above, i.e. screws within 200mm of each end of each dummy rafter section and at 1.000m maximum centres along the length of each dummy rafter.

(iv) DOUBLE 90 X 45 DUMMY RAFTERS

Where the design chart requires dummy rafters at centers less than 450mm it is permissible to use double dummy rafters at twice the spacing, e.g. instead of 90 x 45 dummy rafters at 300mm centers, can use double 90 x 45 dummy rafters (on edge) at 600mm centers. Double dummy rafters should be nailed together (side by side) with 90 x 3.55mmø FH galvanized nails at 300mm centers.

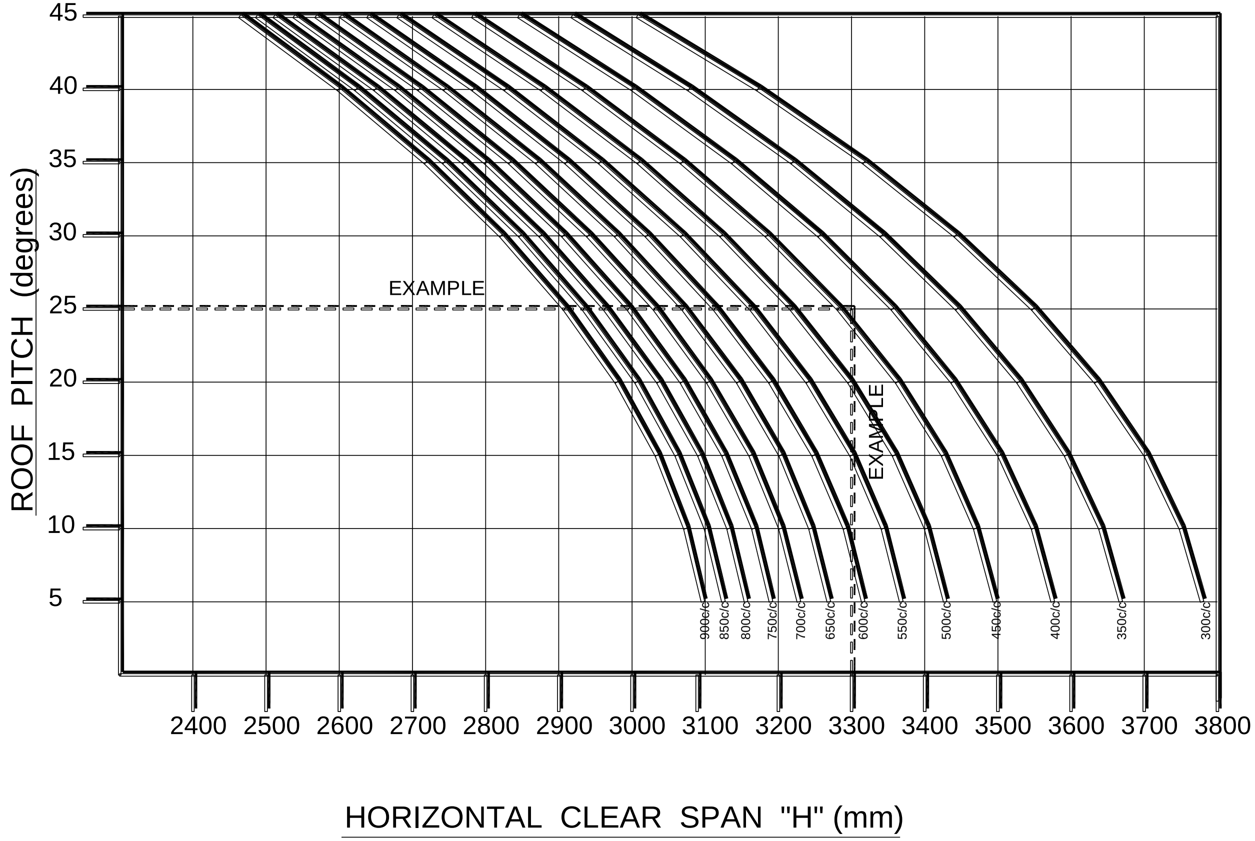
Screw fix double dummy rafters to 35mm sarking centrally through alternate 90 x 45 members along length as Clause 9.1(a), i.e. screws within 200mm of each end of each dummy rafter section and at 1.000m maximum centers along length.

(v) FIXING OF 140 X 45 DUMMY RAFTERS

Ex 150 x 50 dummy rafters to be fixed to sarking with Lumberlok CPC40 (concealed purlin cleats). Locate cleats within 200mm of each end of sarking spans and at 900mm maximum centres both sides of each dummy rafter.
Each CPC40 cleat to be fixed to rafters with 4 nails (30 x 3.15mmϕ FH galvanised), and to sarking with 2/10 gauge x 30mm wood screws, class 2 minimum durability rating, or class 3 or class 4.

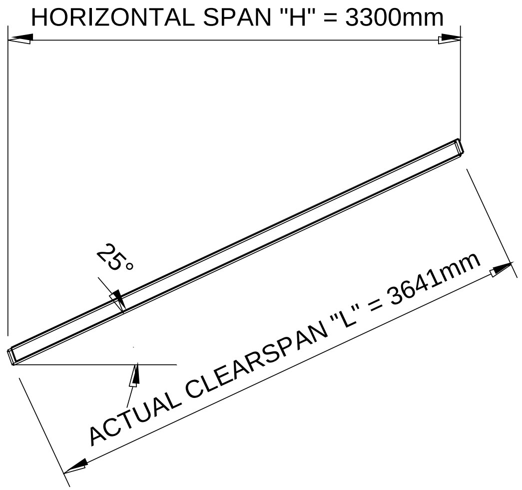
CPC40 cleats may also be used to fix 90 x 45 and double 90 x 45 dummy rafters, as specified above.

90x45 DUMMY RAFTER SELECTION CHART (35mm sarking)



EXAMPLE:

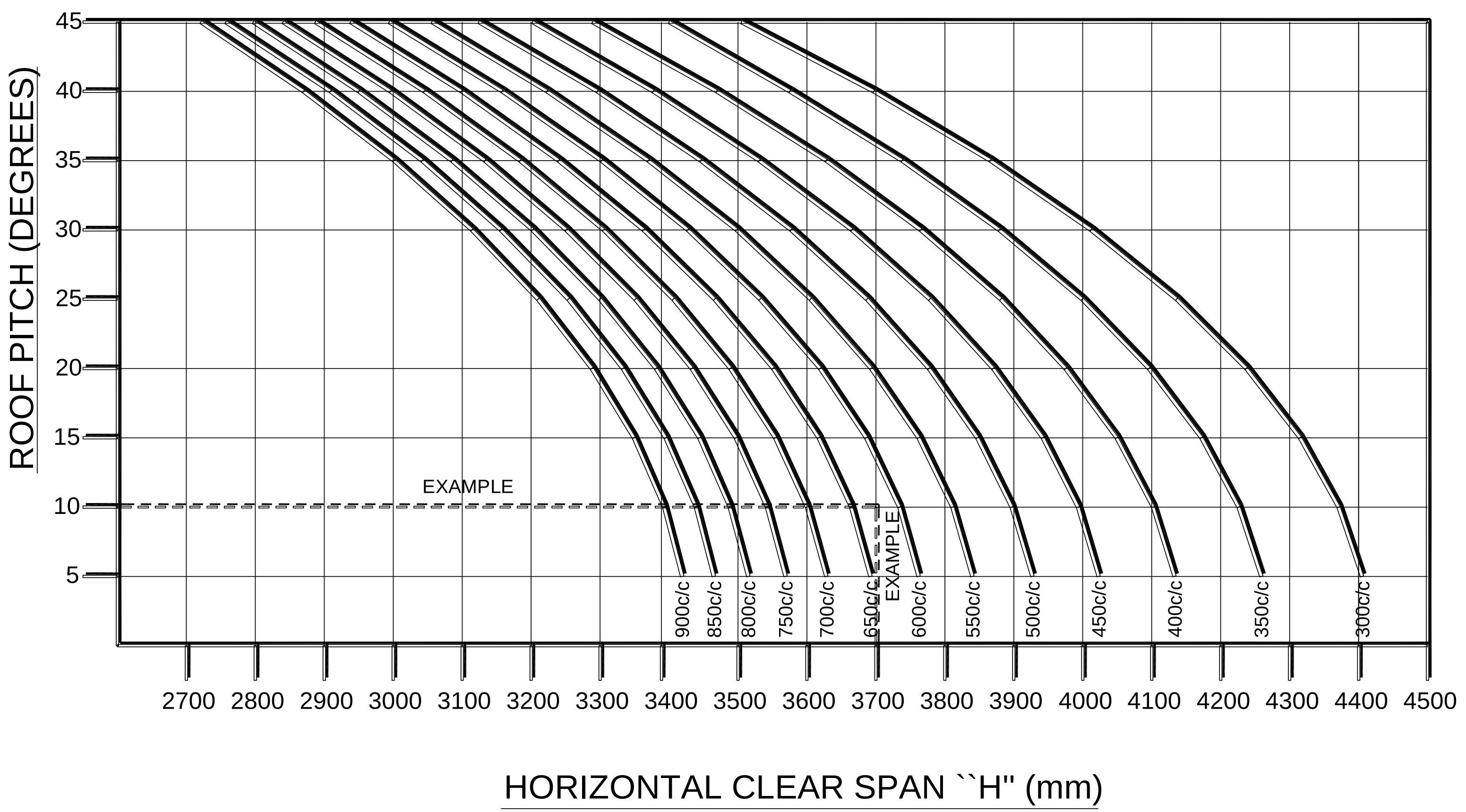
A HORIZONTAL CLEAR SPAN OF 3300mm AT 25° REQUIRES 90x45 DUMMY RAFTERS AT 400mm CRS. MAXIMUM LONG TERM DEFLECTION OF 0.006 L = 21.8mm

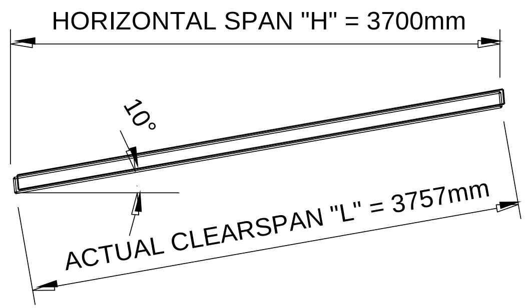


NOTE:

DUMMY RAFTER CENTRES ARE TABULATED FOR 90x45, MSG8 OR VSG8, RADIATA PINE DUMMY RAFTERS, AND A LONG TERM DEFLECTION LIMITATION OF 0.006 L $\left(\frac{SPAN}{167}\right)$

140x45 DUMMY RAFTER SELECTION CHART (35mm SARKING)



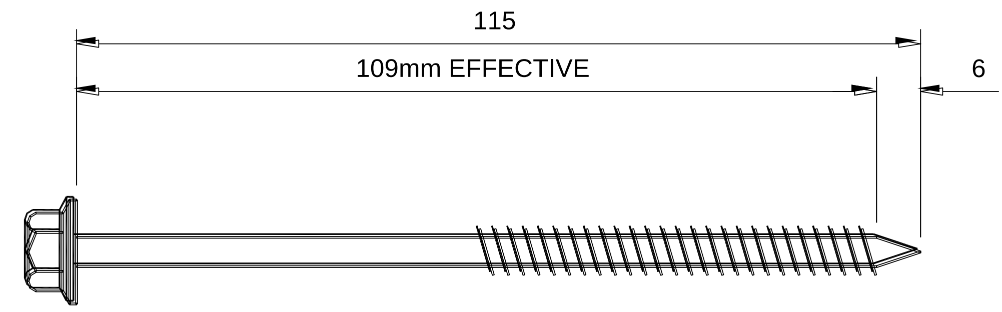


NOTE:

DUMMY RAFTER CENTRES ARE TABULATED FOR 140x45, MSG8 OR VSG8, RADIATA PINE DUMMY RAFTERS, AND A LONG TERM DEFLECTION LIMITATION OF 0.0033L $\left(\frac{SPAN}{300}\right)$

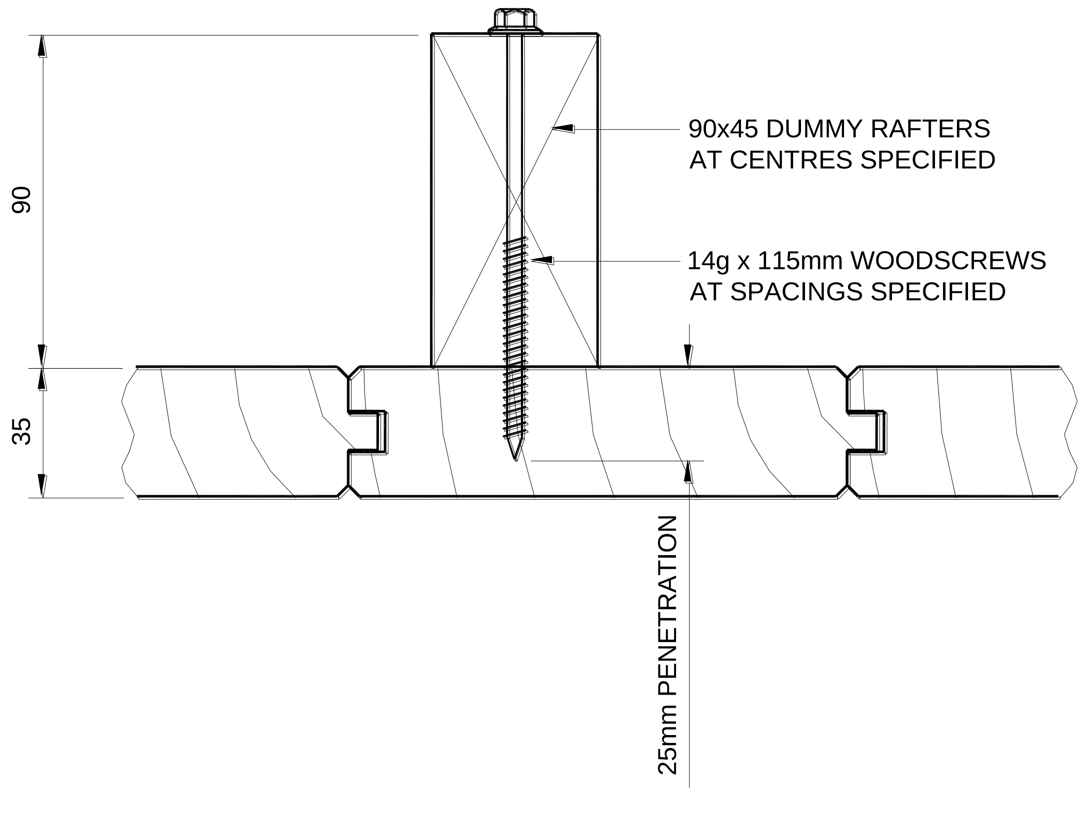
EXAMPLE:

A HORIZONTAL CLEAR SPAN OF 3700mm AT 10° REQUIRES 140x45 DUMMY RAFTERS AT 600mm CRS. MAXIMUM LONG TERM DEFLECTION OF $\frac{SPAN}{300}$ = 12.5mm

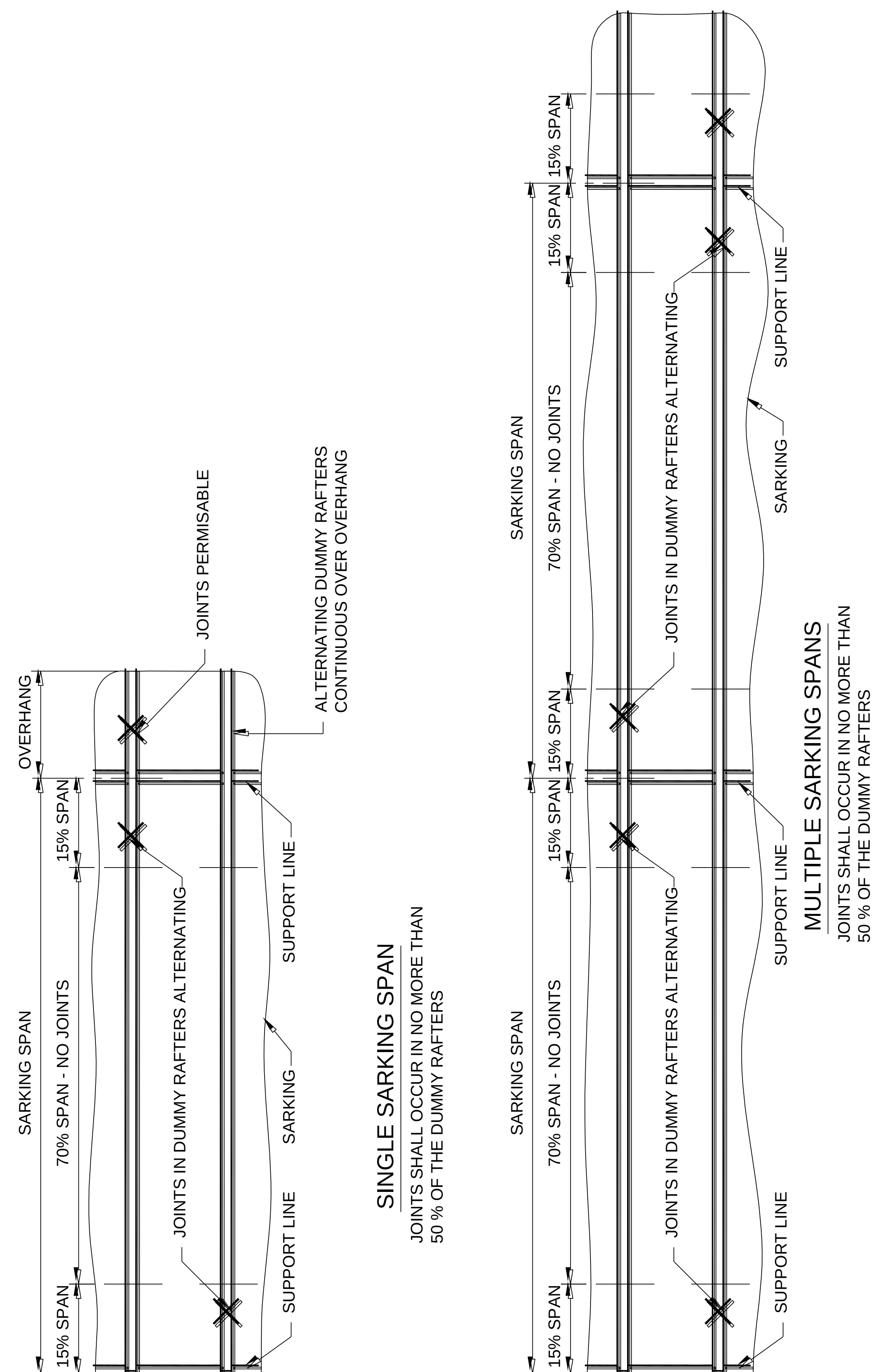


FORTRESS 14g x 115mm 1:1

TYPE 17 HEXAGONAL WASHER FACE



TYPICAL 90x45 DUMMY RAFTER FIXING 1:2


OVERHANG
SARKING SPAN
15% SPAN
70% SPAN - NO JOINTS
15% SPAN
JOINTS PERMISABLE
JOINTS IN DUMMY RAFTERS ALTERNATING
ALTERNATING DUMMY RAFTERS CONTINUOUS OVER OVERHANG
SUPPORT LINE
SARKING
SUPPORT LINE
RIDGE
SINGLE SARKING SPAN
JOINTS SHALL OCCUR IN NO MORE THAN 50 % OF THE DUMMY RAFTERS
SARKING SPAN
SARKING SPAN
15% SPAN
15% SPAN
70% SPAN - NO JOINTS
15% SPAN
15% SPAN
70% SPAN - NO JOINTS
15% SPAN
JOINTS IN DUMMY RAFTERS ALTERNATING
JOINTS IN DUMMY RAFTERS ALTERNATING
SUPPORT LINE
SARKING
SUPPORT LINE
SUPPORT LINE
RIDGE
MULTIPLE SARKING SPANS
JOINTS SHALL OCCUR IN NO MORE THAN 50 % OF THE DUMMY RAFTERS

8. FASCIA BEAM SPANS AND SUPPORT POST FOUNDATION SIZES

The following Table gives maximum fascia beam spans (maximum post spacings) for various sarking spans (supported by the fascia beam) and for different wind zones (in terms of NZS3604:1999).

The maximum post spacings are given based on a standard 235 x 32 Lockwood GL8 Grade laminated fascia beam.

Sarking Span (metres)	Maximum Post Spacing (metres	Design Wind Speed			
		50m/s Very High	44m/s High	37m/s Medium	32m/s Low
1.2	4.200	0.231m^3 (700x700x500)	0.168m^3 (600x600x500)	0.105m^3 (500x500x500)	0.066m^3 (500x500x300)
1.5	4.000	0.278m^3 (700x700x600)	0.203m^3 (600x600x500)	0.127m^3 (500x500x500)	0.081m^3 (500x500x400)
1.8	3.800	0.319m^3 (700x700x700)	0.233m^3 (700x700x500)	0.142m^3 (600x600x400)	0.095m^3 (500x500x400)
2.1	3.600	0.354m^3 (700x700x700)	0.259m^3 (700x700x600)	0.172m^3 (600x600x500)	0.106m^3 (500x500x500)
2.4	3.400	0.386m^3 (800x800x600)	0.283m^3 (700x700x600)	0.184m^3 (600x600x500)	0.119m^3 (500x500x500)
2.7	3.300	0.420m^3 (800x800x700)	0.308m^3 (700x700x700)	0.196m^3 (700x700x400)	0.128m^3 (500x500x500)
3.0	3.200	0.453m^3 (800x800x700)	0.332m^3 (700x700x700)	0.212m^3 (700x700x500)	0.138m^3 (600x600x400)

Note that the concrete volumes shown above assume 20 MPa minimum grade concrete. Normal density 24 kN/m^3. The sizes shown are for the maximum post spacing indicated.

For lesser posts spacings the concrete volumes may be reduced proportionately. For example for the 50m/s design condition, 1.2m sarking span and posts spaced at 2.8m c/c, the volume of concrete at each internal post is given by

$$V = 0.231 \times \frac{2.8}{4.2} \mathrm{m}^3 = 0.154\mathrm{m}^3$$

At posts at ends of spans the concrete volumes may be halved.

In the shaded area the fixing at the top of the post on to the 235 x 32 fascia requires two Pryda Multigrips between the post and the back fence of the fascia (above the sarking), in addition to the 2 M10 galvanised bolts. Refer Lockwood Detail D-25.

Alternatively provide 6 nails (100mm x 4ø galv.) through the fascia into the post, in addition to the 2/M10 galvanised bolts.

9. TIE RODS

SCOPE

This section sets out the minimum requirements for tie rods to provide structural tie down and for bracing stability in Lockwood walls, based on the following limitations:
(Designs exceeding these limits should be the subject of specific engineering design.)

- 35mm sarking plus 90 x 45 dummy rafters at 900mm c/c (spans requiring larger dummy rafters or dummy rafters at closer centres to be the subject of specific engineering design).
- A “Light” roof and “Very High Wind”, both in terms of NZS 3604:1999.
- Maximum 7.2m roof truss span (no allowance has been made for girder truss point loadings).
- Maximum 0.6m eaves width.
- Maximum 1.5m verandah roof width.
- Maximum 45º roof pitch.

Tie rods shall be provided in the following positions:

1. One tie rod adjacent to each external corner – or one tie rod in the corner profile.

2. One tie rod each side of each b or b repair profile positions.

3. One tie rod at each end of each bracing panel used in bracing unit calculations.
   - double tie rods in special panels, e.g. Panel No. 16 (1.786m ply faced panel).

4. One tie rod adjacent to each exterior wall opening. Spacing not to exceed limits given in Clauses 5 to 10 below (refer also Lintel Design section).

5. On 62mm exterior walls – (sarked roof systems), at 2.7m maximum c/c, or 3.3m maximum c/c where top board stiffened by 140 x 45 E beam (refer Lintel design section for E beam nailing specifications).

6. On 62mm exterior walls – (trussed roof systems), at 2.1m maximum c/c or 2.7m maximum c/c where top two boards bolted together at mid span (maximum 12mmø holes drilled for bolts).

7. On 44mm exterior walls (sarked roof systems), at 2.2m maximum c/c, or 3.2m maximum c/c where the top two boards are nail plated together (refer Lintel design section for nail plate fixing specifications).

8. On 44mm exterior walls (trussed roof systems), at 1.8m maximum c/c, or 2.5m maximum c/c where the top two boards are nail plated together (refer Lintel design section for nail plate fixing specifications), or 2.9m maximum c/c where the top two boards are nail plated together and stiffened by a 145 x 45 lintel stiffening member (refer Lintel design section for E beam nailing specifications).

9. Adjacent to each internal or external post or stiffener which is supporting a roof beam and which is not effectively tied down at roof level by bolting through the mortice and

tenon and at floor level by effective bolting (Lockwood Standard Detail B-19).

10. Internal 44mm walls supporting roof loads, at 2.6m maximum centres.
A 140 x 45 E beam to be provided for fixing of sarking planks. E beam nailed to wall with 75 x 3.15mmø jolt head galvanised nails at 180mm c/c (Maximum), staggered up and down along length. Nail each sarking plank through into top of E beam with 2/100 x 4.0mmø flat head galvanised nails.

11. In any case where the top board of an external side wall (supporting roof loads) is not a complete board or does not fit within the above criteria, the tie rod spacing should be the subject of specific engineering design.

10. LINTEL DESIGN

SCOPE

All lintel design has been based on the following limitations:
(Designs exceeding these limits should be the subject of specific engineering design.)

- 35mm sarking plus 90 x 45 dummy rafters at 900mm c/c (spans requiring larger dummy rafters or dummy rafters at closer centres to be the subject of specific engineering design).
- A "Light" roof and "Very High Wind", both in terms of NZS 3604:1999.
- Maximum 7.2m roof truss span (no allowance has been made for girder truss point loadings).
- Maximum 0.6m eaves width.
- Maximum 1.5m verandah roof width.
- Maximum 45º roof pitch.

A SIDE WALLS – ALL JOINERY EXCLUDING BI-FOLDS

1. <u>62mm Wall System, Sarked Roof</u>

(a) One board lintel is adequate for clear spans up to 1.866m (2 light windows). Tie rods at a maximum spacing of 2.7m c/c.

(b) One board lintel plus 180 x 45 special E beam is adequate for clear spans up to 2.752m (3 light window).
Tie rods at a maximum spacing of 3.3m.
Top of E beam angle cut to sarking pitch. Nailing 75mm x 3.15mmø jolt head galvanised nails at 200mm c/c staggered.

2. <u>62mm Wall System, Trussed Roof</u>

(a) A one board lintel is adequate for clear spans up to 1.866m (2 light window). Tie roads at a maximum spacing of 2.1m.

(b) A two board lintel (bolted) plus 140 x 45 E beam is adequate for clear spans up to 2.752m (3 light window).
Tie rods at a maximum spacing of 3.1m c/c.
Bolting – one M10 galvanised bolt centrally on lintel span. Holes drilled 12mm maximum diameter.
140 x 45 E beam nailed to wall boards with 90 x 3.55ø jolt head galvanised nails at 200mm c/c staggered.

3. <u>44mm Wall System, Sarked Roof</u>

(a) One board lintel is adequate for clear spans up to 1.866m (2 light window). Tie rods at a maximum spacing of 2.2m c/c.

(b) A two board lintel, nail plated together is adequate for clear spans up to 2.752m (3 light window).

Tie rods at a maximum spacing of 3.2m.
Under each truss on exterior face of 44mm wall boards provide a 6N10 (190mm x 76mm) Pryda knuckle nail plate centrally over the joint between the top two boards (190mm dimension oriented vertically).

4. <u>44mm Wall System, Trussed Roof</u>

(a) A two board lintel nail plated together is adequate for clear spans up to 1.866m (2 light window).
Tie rods at a maximum spacing of 2.5m.
Nail plating as 3(b) above.

(b) A two board lintel nail plated together (as 3(b) above), plus a 190 x 45 lintel stiffening is adequate for clear spans up to 2.752m (3 light window).
Tie rods at a maximum spacing of 3.1m c/c.

Ex 200 x 50 (190 x 45 finished dimension), VSG8 or MSG8 Structural Grade (minimum) Radiata Pine, within batten space. Ex 200 x 50 nailed to 14th board hard up under top plate with 75 x 3.15mmø FH galvanised nails at 120mm c/c staggered up and down along length.
Nail through top plate into top of ex 200 x 50 with 100 x 4.0mmø FH galvanised nails at 300mm c/c.

B END WALLS AND GABLE END WALLS – ALL JOINERY EXCLUDING BI-FOLDS

62mm and 44mm wall systems, sarked and trussed roofs.

(a) Point loads from beams the subject of special consideration.

(b) A one board (minimum) lintel is adequate for clear spans up to 2.752m. Tie rods at a maximum spacing of 3.0m c/c.

(c) A two board (minimum) lintel is adequate for clear spans up to 3.070m. Tie rods at a maximum spacing of 3.200m c/c
44mm wall boards to be nail plated together at 1.2m maximum centres (similar to A 3(b) above).
62mm wall boards to be bolted (as in A 2(b) above)

(d) A five board minimum lintel is adequate for clear spans up to 4.00m.
All 44mm wall boards to be nail plated together at 1.2m maximum centres (similar to A 3(b) above).
62mm wall boards to be bolted (as in A 2(b) above)

C SIDE WALLS – BIFOLD JOINERY

Note: All lintel flitch plate options as given below to be the subject of specific engineering design and detailing

1. 62mm Wall System, Sarked Roof

(a) One board lintel is adequate for clear spans up to 1.570m (FB doors).
Tie rods at a maximum spacing of 2.7m.

(b) One board lintel plus 140 x 45 E beam is adequate for clear spans up to 1.866m (2 light windows).
Tie rods at a maximum spacing of 2.7m c/c.
Top of E beam angle cut to sarking patch. Nailing 75mm x 3.15mmø JH galvanised nails at 200mm c/c staggered.

(c) One board lintel plus ex 250 x 50 special E beam plus 130mm x 6mm steel plate screw fixed to the inside face of 62mm top board, is adequate for clear spans up to 2.752m.
Tie rods at a maximum spacing of 3.1m c/c.

(d) One board lintel plus ex 250 x 50 special E beam plus 150mm x 10mm steel plate screw fixed to the inside face of the 62mm top board is adequate for clear spans up to 3.07m.
Tie rods at a maximum spacing of 3.4m c/c.

2. 62mm Wall System, Trussed Roof

(a) A two board lintel Bolted together at mid-span (as A2(b)) is adequate for clear spans up to 1.866m.
Tie rods at a maximum spacing of 2.6m c/c.

(b) A two board lintel plus 240 x 45 E Beam is adequate for clear spans up to 2.370m (FB3 doors).
Tie rods at a maximum spacing of 2.6m c/c.
Top of E beam angle cut to sarking patch. Nailing 75mm x 3.15mmø JH galvanised nails at 200mm c/c staggered.

(c) A two board lintel bolted together at mid-span (as A2(b)) and stiffened by an ex 300 x 50 (290 x 45 finished dimension) special E beam nailed to the top boards, is adequate for clear spans up to 2.752m (3 light window).
Tie rods at a maximum spacing of 3.1m.
290 x 45 E beam nailed to wall boards with 90 x 3.55mmø JH galvanised nails at 300mm c/c top and bottom.

3. 44mm Wall System, Sarked Roof

(a) A two board lintel nail plated together (as in A 3(b) above), plus a 140 x 45 lintel stiffening is adequate for clear spans up to 1.866 (2 light window).
Tie rods at a maximum spacing of 3.2 c/c.

Ex 150 x 50 (140 x 45 finished dimension), VSG8 or MSG8 Structural Grade (minimum) Radiata Pine lintel stiffening, within batten space. 140 x 45 nailed to 14th board hard up under top plate with 75 x 3.15mmø FH galvanised nails at 120mm c/c staggered up and down along length.
Nail through top plate into top of ex 150 x 50 with 100 x 4.0mmø FH galvanised nails at 300mm c/c.

(b) A two board lintel nail plated together (as in A 3(b) above), plus a 240 x 45 lintel stiffening is adequate for clear spans up to 2.752m (3 light window).
Tie rods at a maximum spacing of 3.9m c/c.
Ex 250 x 50 (240 x 45 finished dimension), VSG8 or MSG8 Structural Grade (minimum) Radiata Pine lintel stiffening within batten space (nailed to boards as in C 3(a) above).

(c) A two board lintel nail plated together (as in A 3(b) above), plus a 290 x 45 lintel stiffening is adequate for clear spans up to 3.07m (FB4 doors).
Tie rods at a maximum spacing of 3.9m c/c.
Ex 300 x 50 (290 x 45 finished dimension), VSG8 or MSG8 Structural Grade (minimum) Radiata Pine lintel stiffening within batten space (nailed to boards as in C 3(a) above).

4. <u>44mm Wall System, Trussed Roof</u>

(a) A two board lintel nail plated together (as in A3 (b)) is adequate for clear spans up to 1.866m (2 light window).
Tie rods at a maximum spacing of 2.5m c/c.

(b) A two board lintel nail plated together (as A3 (b) above), plus a 240 x 45 lintel stiffening is adequate for clear spans up to 2.320m (FB3 doors).
Tie rods at a maximum spacing of 2.6m c/c.

Ex 250 x 50 (240 x 45 finished dimension), VSG8 or MSG8 Structural Grade (minimum) Radiata Pine, within batten space. Ex 250 x 50 nailed to 14th board hard up under top plate with 75 x 3.15mmø FH galvanised nails at 120mm c/c staggered up and down along length.
Nail through top plate into top of ex 250 x 50 with 100 x 4.0mmø FH galvanised nails at 300mm c/c.

(c) A two board lintel nail plated together (as in A3 (b) above), plus a 290 x 45 lintel stiffening is adequate for clear spans up to 2.752m (3 light window).
Tie rods at a maximum spacing of 3.2m c/c.
Ex 300 x 50 (290 x 45 finished dimension), VSG8 or MSG8 Structural Grade (minimum) Radiata Pine within batten space.
Ex 300 x 50 nailed to boards hard up against top plate, with 75 x 3.15mmø FH galvanised nails at 240mm c/c along length, top and bottom.
Nail through top plate into top of ex 300 x 50 with 100 x 4.0mmø FH galvanised nails at 300mm c/c.

D END WALLS AND GABLE END WALLS – BIFOLD JOINERY

62mm and 44mm wall systems, sarked and trussed roofs.

(a) Point loads from beams the subject of special consideration.

(b) A one board (minimum) lintel is adequate for clear spans up to 1.866m. Tie rods at a maximum spacing of 2.6m c/c.

(c) A two board (minimum) lintel is adequate for clear spans up to 2.752m. Tie rods at a maximum spacing of 3.200m c/c.
44mm wall boards to be nail plated together at 1.2m maximum centres (similar to A 3(b) above).
62mm wall boards to be bolted (as in A 2(b) above)

(d) A five board minimum lintel is adequate for clear spans up to 4.00m.
All 44mm wall boards to be nail plated together at 1.2m maximum centres (similar to A 3(b) above).
62mm wall boards to be bolted (as in A 2(b) above)

10.1 SOLID 97mm & 107mm BOARD LINTELS

DESCRIPTION

There are three types of solid top boards, a pine board, an aluminium clad board and a cedar clad board.
All board types are vertically laminated Radiata Pine. All laminates are machine stress graded to ensure that a minimum modulus of elasticity of 8.0GPa is achieved.

SCOPE

All lintel design has been based on the following limitations:
(Designs exceeding these limits should be the subject of specific engineering design).

(a) SARKED HIP OR GABLE ROOF (REFER TABLE 1)

- 35mm sarked roof with either 90 x45 or 140 x 45 dummy rafters
- Maximum 25 degree roof pitch
- A maximum horizontal sarking span supported by lintel of 3.2m and a maximum 0.6m eaves width (2.2m loaded horizontal dimension)
- A “light” roof and “Very High Wind” in terms of NZS 3604:1999.

(b) TRUSSED ROOF (REFER TABLE 2)

- A “light” roof and “Very High Wind” in terms of NZS 3604:1999
- Maximum 25 degree roof pitch
- Maximum 7.2m roof truss span (no girder truss point loads)
- Maximum 0.6m eaves width

TABLE 1: SARKED HIP OR GABLE ROOF

The following table gives lintel types required for Lockwood sarked roofs when using solid 97mm or solid 107mm top boards, for selected opening sizes and subject to limitations given in scope 10.1 (a) above.

JOINERY OPENING (mm)	**SOLID 97mm or 107mm TOP BOARD LINTEL**
ALL JOINERY EXCLUDING BI-FOLDS	
1866	1 Board
2752	1 Board
3710	1 Board + Ex 190 x 45 E-beam
BI-FOLD JOINERY	
2320	1 Board
2752	1 Board + Ex 190 x 45 E-Beam
3070	1 Board + 130 x 10mm FL*

* FL stands for mild steel flat plate (flitched top board lintel).

TABLE 2: TRUSSED ROOF

The following table gives lintel types required for trussed roofs when using solid 97mm or solid 107mm top boards, for selected opening sizes and subject to limitations given in scope 10.1 (b) above.

JOINERY OPENING (mm)	**SOLID 97mm or 107mm TOP BOARD**
ALL JOINERY EXLUDING BI-FOLDS	
1866	1 Board
2752	1 Board
3710	1 Board + Ex 130 x 10mm FL*
BI-FOLD JOINERY	
2320	1 Board + Ex 190 x 45 E-beam
2752	1 Board + Ex 130 x 10 FL*
3070	1 Board + Ex 150 x 10mm FL*

* FL stands for mild steel flat plate (flitched top board lintel).

CHANGES TO SECTION C - LATERAL STABILITY IN THE LOCKWOOD SYSTEM

SECTION C LATERAL STABILITY – SUPPORTING INFORMATION

This clause replaces Section C. LATERAL STABILITY IN THE LOCKWOOD SYSTEM. Pages C-1 to C-42, in its entirety.

1. BRACING TEST REPORT SUMMARY

The lateral wall bracing testing conducted by Holmes Solutions in 2013 is summarised here. Full test results are available on request.

Testing was completed on 16 Lockwood walls in total, 7 of these were internal walls with a thickness of 44mm and 9 were external walls with a thickness of 107mm. Each tested wall was solid in construction, with no window, door, or other openings. All walls were assembled as per Lockwood standard practice, unless noted otherwise.

Testing was completed in accordance with the BRANZ EM3-V3 testing protocol with the displacement targets modified to reflect the increased displacement allowance in AS / NZS 1170.1 for non-plaster lined walls. A summary of the testing performed on the walls is shown in Table C.1. All walls tested were 2.4 m in height. Figure C.1 shows the test apparatus used for testing.

At the completion of the testing, calculations of the Bracing units (BU) capacity of the tested walls were made in accordance to EM3-V3 analyses, with revised F1 and F2 factors to reflect the robustness of the Lockwood wall systems.

A number of the wall elements were subjected to additional cycles of testing, achieving displacement cycles in excess of 200 mm with no significant reduction in load carrying capacity. Furthermore, additional tensile testing was completed on individual tie rod components.

Key findings from the testing are as follows:

1. Tie rods are critical to the bracing capacity of a walls section. To ensure a section of wall can withstand loading from either end (in both push and pull directions) a tie rod is required to be installed within 150mm of each end of the section of wall.
2. Aluminium profiles contribute towards the elasticity of walls. After displacement, one or more aluminium profiles act to provide a degree of elastic recovery and act to straighten the wall as the loading is removed.

An overview of the testing and results are given below in Figures C.1, C.2 and C.3 and Tables C.1, C.2 and C3.

Figure C.1 – Test apparatus and test specimen

Table C.1 – Test Matrix

ID	Wall Length	X-Profiles	Tie rods	Wall type
LW1	1.2	2	2	Internal
LW2	1.2	2	2	Internal
LW3	1.2	1	1	Internal
LW4	1.2	1	1	Internal
LW5	2.4	2	2	Internal
LW6	4.8	2	2	Internal
LW7	4.8	2	2	Internal
LW8	1.2	2	2	External
LW9	1.2	2	2	External
LW10	2.4	2	2	External
LW11	2.4	0	2	External
LW12	2.4	2	2	External
LW13	2.4	0	2	External
LW14	2.4	1	2	External
LW15	2.4	1	2	External
LW16	4.8	2	2	External

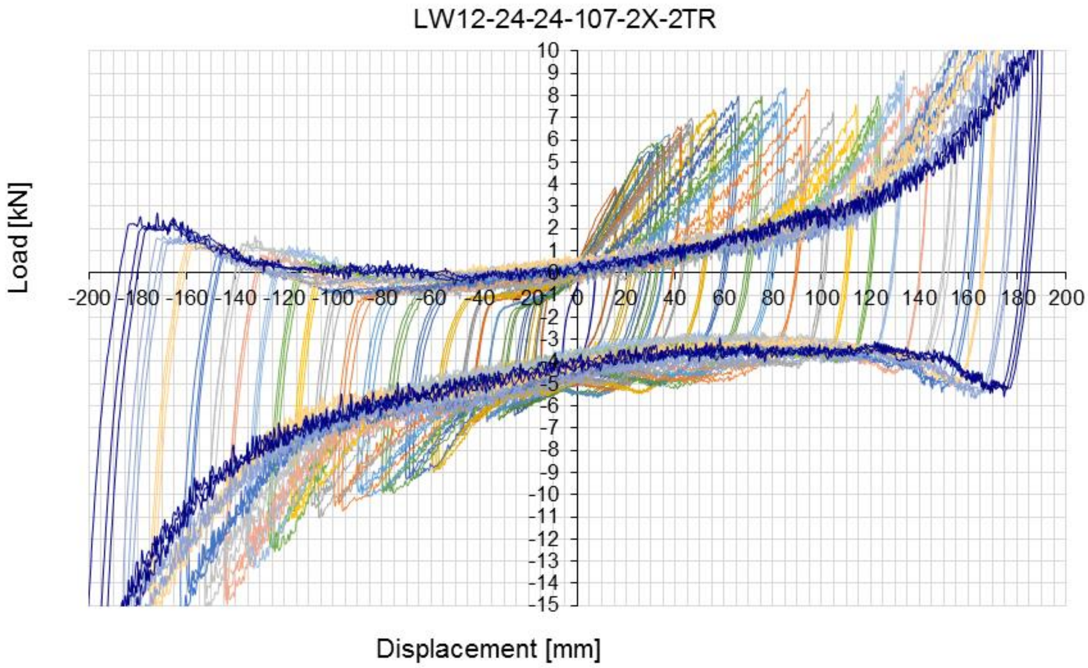


Figure C.2 – Typical load-displacement hysteresis graph

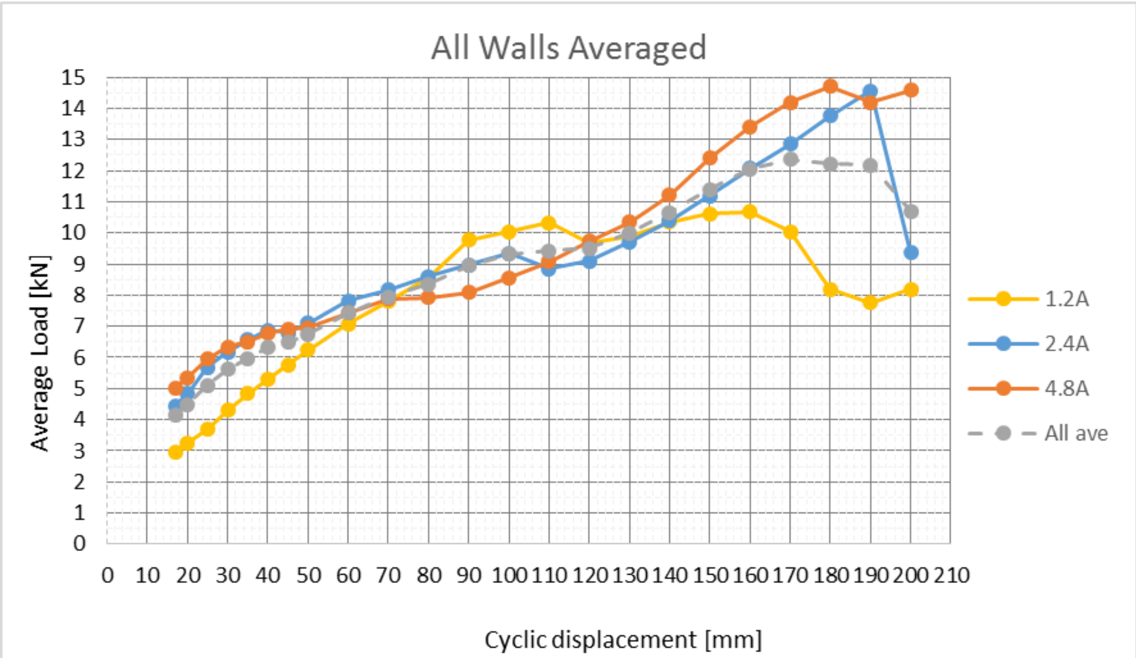


Figure C.3 – Average load versus cyclic displacement for the average of all wall lengths

Table C.2 – Detailed Results from EM3-V3 Analysis for Internal Walls

ID	Length	X-Profile	Tie Rods	EQ Δ	BR EQ (0.381)	EQ Δ ave	BR EQ ave	Lesser ave [BU]	BU/m
LW1	1.2	2	2	236.0	182.5	195.5	151.4	151.4	126.2
LW2		2	2	155.0	120.3				
LW5	2.4	2	2	239.1	278.0	239.1	278.0	239.1	99.6
LW6	4.8	2	2	188.4	250.4	212.8	253.0	212.8	44.3
LW7		2	2	237.1	255.7				
LW3	1.2	1	1	159.7	132.3	144.5	99.4	99.4	82.8
LW4		1	1	129.3	66.5				

ID	Length	X-Profile	Tie Rods	Wind Δ	BR Wind (0.563)	Wind Δ ave	BR wind ave	Lesser ave [BU]	BU/m
LW1	1.2	2	2	244.8	123.5	204.6	102.5	102.5	85.4
LW2		2	2	164.3	81.4				
LW5	2.4	2	2	266.4	188.1	266.4	188.1	188.1	78.4
LW6	4.8	2	2	195.8	169.4	221.9	171.2	171.2	35.7
LW7		2	2	248.1	173.0				
LW3	1.2	1	1	167.8	89.5	149.7	67.3	67.3	56.1
LW4		1	1	131.6	45.0				

Table C.3 – Detailed Results from EM3-V3 Analysis for External Walls

ID	Length	X-Profile	Tie Rods	EQ Δ	BR EQ (0.381)	EQ Δ ave	BR EQ ave	Lesser ave	BU/m
LW8	1.2	2	2	183.0	138.1	209.0	160.3	160.3	133.6
LW9		2	2	235.1	182.5				
LW10	2.4	2	2	182.5	199.3	206.5	211.8	203.4	84.7
LW12		2	2	230.5	224.3				
LW16	4.8	2	2	210.1	283.6	210.1	283.6	210.1	43.8
LW11	2.4	0	2	86.0	80.3	178.6	166.7	166.7	69.4
LW13		0	2	271.2	253.0				
LW14	2.4	1	2	208.5	260.3	175.1	225.8	175.1	73.0
LW15		1	2	141.8	191.3				

ID	Length	X-Profile	Tie Rods	Wind Δ	BR wind (0.563)	Wind Δ ave	BR wind ave	Lesser ave	BU/m
LW8	1.2	2	2	195.6	93.4	220.0	108.5	108.5	90.4
LW9		2	2	244.4	123.5				
LW10	2.4	2	2	194.0	134.8	218.7	143.3	143.3	59.7
LW12		2	2	243.3	151.8				
LW16	4.8	2	2	223.9	191.9	223.9	191.9	191.9	40.0
LW11	2.4	0	2	85.4	54.4	174.8	112.8	112.8	47.0
LW13		0	2	264.3	171.2				
LW14	2.4	1	2	213.9	176.1	182.4	152.8	152.8	63.7
LW15		1	2	150.9	129.4				

2. DUCTILITY OF THE LOCKWOOD SYSTEM

The Lockwood wall panel system show high levels of ductility, where ductility is defined as the ability to undergo inelastic deformations with little or no reduction in load carrying capacity. The load deformation responses obtained for the tested walls showed very little reduction in load carrying capacity up till lateral displacements in excess of 200 mm.

The Serviceability Limit State (SLS) of a Lockwood wall is limited in AS/NZS 1170 to an allowable displacement at the top of the wall of 16 mm (for a 2400 mm wall height). Under the Ultimate Limit State (ULS) the maximum allowable displacement of a wall must be limited so as to prevent excessive deflections adversely affecting the non-structural elements in the building envelop. For a Lockwood wall, it was determined that the ULS deflections should be limited to less than 2.5% of wall height, a limitation typically utilised in the commercial building sector. For a 2400 mm wall height, this equates to a displacement limit of 60 mm at the top of the wall.

For all Lockwood walls tested, it was observed that the walls continued to gain strength after the attainment of the 60 mm wall displacement limits. For a number of walls tested until failure, the peak strength was maintained until wall displacements in excess of 200 mm. This additional wall strength ensures the Lockwood walls have a high level of ductility, however this additional strength gain after the attainment of 60 mm could not be used in assessing the Brace Capacity of the walls due to the deflection limitations imposed.

The failure mode observed in all testing completed was ductile in nature. No components underwent brittle failure and all walls maintained a high degree of lateral bracing strength and significant vertical load carrying capacity at the point of failure. This high level of ductility in the system provides significant confidence in the lateral bracing capacity of the Lockwood walls, however does not directly add to the bracing capacity achieved by the wall system.

3. VERIFICATION OF LOCKWOOD ROOF AND WALL WEIGHTS IN TERMS OF NZS 3604:2011

(i) VERIFICATION OF LOCKWOOD 35mm SARKED ROOF SYSTEM AS A LIGHT ROOF IN TERMS OF NZS 3604:2011

Weight of NZS 3604 Light Roof

- NZS 3604:2011 Light Roofing (by definition) 20.0 kg/m2
- Purlins 4.0 kg/m^2
- Trusses @ 900mm c/c 9.0 kg/m^2
- Ceiling battens (75 x 40 x 400mm c/c) 4.0 kg/m^2
- 10mm Gib ceiling 6.8 kg/m^2

Total weight of Light Roof in Terms of NZS3604:2011 = 43.8 kg/m^2

Weight of 35mm Sarked Lockwood Roof

- Sarking 21.0 kg/m^2
- Dummy Rafters (150 x 50 @ 300mm c/c) max. case 12.6 kg/m^2
- Purlins 2.1 kg/m^2

- Roofing 8.0 kg/m^2

Total weight of 35mm sarked Lockwood roof = 43.7 kg/m^2

Therefore total weight of the 35mm Lockwood sarked roof is less than that of a "Light Roof" in terms of NZS3604:2011.

(ii) VERIFICATION OF LOCKWOOD EXTERIOR WALLS AS LIGHT WALLS IN TERMS OF NZS3604:2011

Weight of NZS 3604 Light Wall

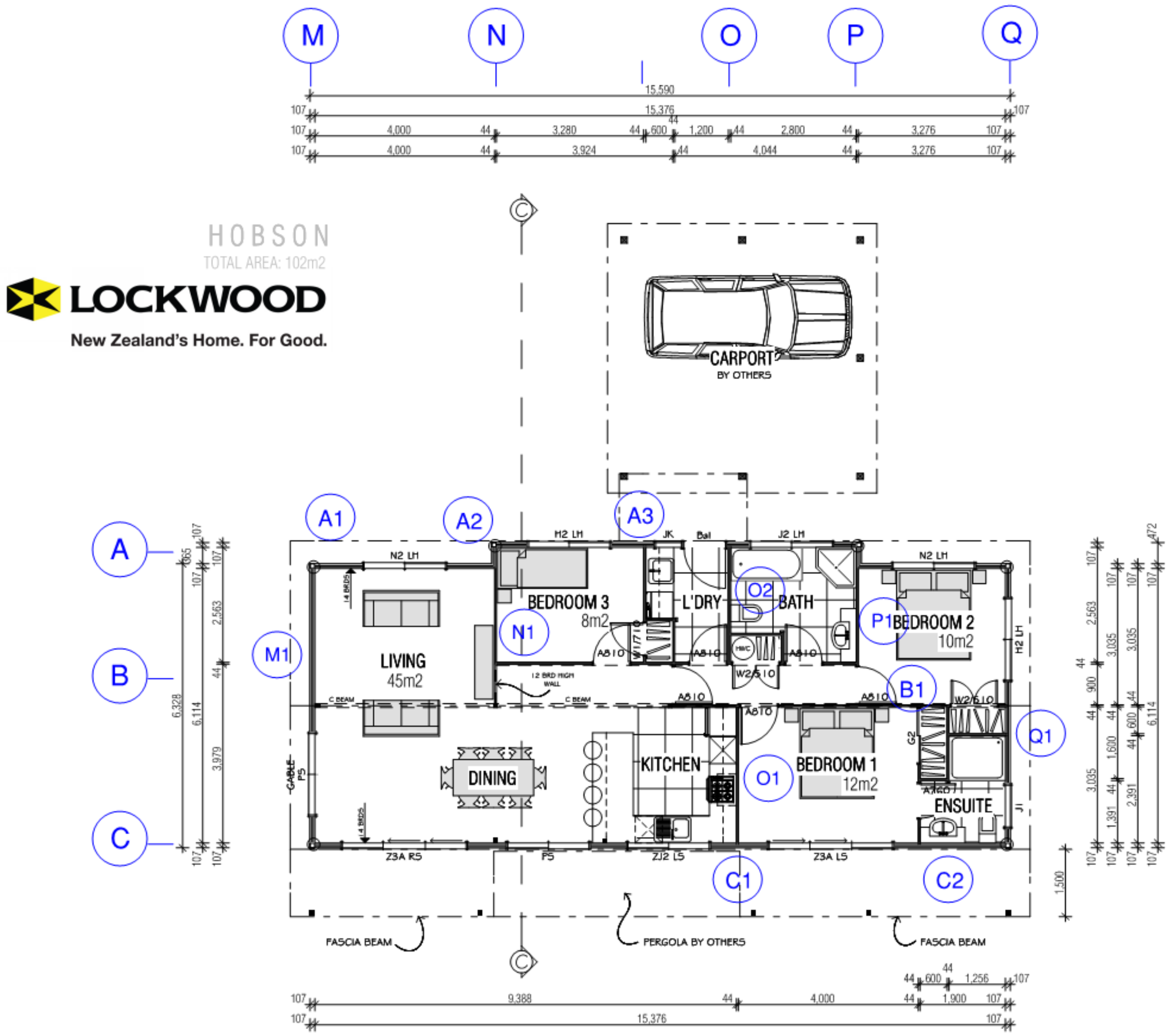
- NZS 3604:2011 Light Wall cladding (by definition) 30.0 kg/m2
- 100 x 50 Framing @ 450 c/c 7.0 kg/m^2
- 10mm Gibralter board 6.8 kg/m^2

- Total weight of Light Wall in terms of NZS3604:2011 43.8 kg/m^2

The total weight of Lockwood 62mm or 97mm walls is approximately 38 kg/m^2.
Therefore total weight of Lockwood exterior wall is less than that of a "Light Wall" in terms of NZS 3604:2011.

4. EXAMPLE BRACING CALCULATION

Name: Hobson (example)
Location of Story: Single
Building height to apex: 4.0m
Stud height: 2.4m
Roof height above eaves: 1.4m
Gross Area: 102 m^2
Average roof pitch: 20 deg
Building length: 15.6m
Building width: 6.5m
Floor type: Concrete Slab
Wind zone: High
EQ zone: 3
Soil Type: D/E



LOCKWOOD WALL BRACING CALCULATION SHEET ALONG

Bracing line			Bracing elements provided					Bracing Achieved			
Line Label	Min B/U's Required (Wind)	Min B/U's Required (EQ)	Bracing Element No	Wall Type	Floor Type	No. of Aluminium Profiles	Wall Section Length (m)	B/U Wind Achieved	B/U E/Q Achieved	Line B/Us Achieved (Wind)	Line B/Us Achieved (EQ)
A	234	234	A1	External 107mm	Concrete	One	1.0	84.8	120	276	390
			A2	External 107mm	Concrete	One	1.0	84.8	120		
			A3	External 107mm	Concrete	Two or more	1.0	106	150		
B	100	102	B1	Internal 44mm	Concrete	Two or more	3.6	148	192	148	192
C	234	234	C1	External 107mm	Concrete	Two or more	1.2	111	171	249	357
			C2	External 107mm	Concrete	Two or more	2.4	138	186		
D										0	0
E										0	0
Total B/U Achieved										673	939
From Sheet A Total B/U Required										380	612

LOCKWOOD WALL BRACING CALCULATION SHEET ACROSS

Bracing line			Bracing elements provided						Bracing Achieved			
Line Label	Min B/U's Required (Wind)	Min B/U's Required (EQ)	Bracing Element No	Wall Type	Floor Type	No. of Aluminium Profiles	Wall height (planks)	Wall Section Length (m)	B/U Wind Achieved	B/U E/Q Achieved	Line B/Us Achieved (Wind)	Line B/Us Achieved (EQ)
M	100	100	M1	External 107mm	Concrete	Two or more	13	3.6	166.0	201	166	201
N	100	100	N1	Internal 44mm	Concrete	Two or more	13	3.0	136.0	182	136	182
O	100	100	O1	Internal 44mm	Concrete	Two or more	13	3.0	136.0	182	261	353
			O2	Internal 44mm	Concrete	Two or more	13	2.4	125.0	171		
P	100	100	P1	Internal 44mm	Concrete	Two or more	13	2.0	117.0	165	117	165
Q	100	100	Q1	External 107mm	Concrete	Two or more	13	2.2	134.0	183	134	183
Total B/U Achieved											814	1084
From Sheet A Total B/U Required											811	612

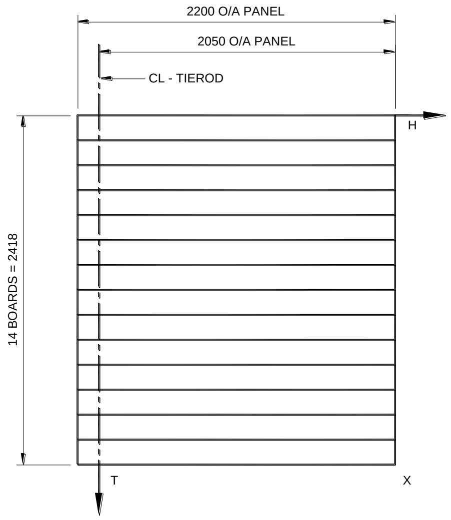
LOCKWOOD

SECTION D UPLIFT ON TIE RODS

1. CASE 1 INTERIOR WALLS For internal Lockwood walls the greatest tie rod tension is required by tie rods preventing wall panels from tipping when the wall panel is stressed to it's full capacity as a bracing element.

From an inspection of the test data the maximum tie rod tension achieved under test loading occurred in the 2.2m O/A length 43mm interior wall panel test No 11.

From the test data, bracing unit capacity achieved was 246.0 B/U. (Limit State)



$$H = \frac{246.0}{20} = 12.30 \ KN$$

by moments about point X

T x 2.050 = H x 2.418

= 12.30 x 2.418

from which

T max = 14.508 KN

UPLIFT ON TIE RODS - INTERIOR WALLS

The standard Lockwood interior tie rod connections are shown on Sheet B21 of the Lockwood Detail Manual.

(a) Check Timber Floor Nog Connections - 150 x 50
Nog - nailed through floor joists at each end with two 100mm x 4φ FH nail plus 4 Pryda Multigrips from nog on to joists. (12 nails per Multigrip - 3.15φ x 30mm FH galv.)

Capacity Loadings

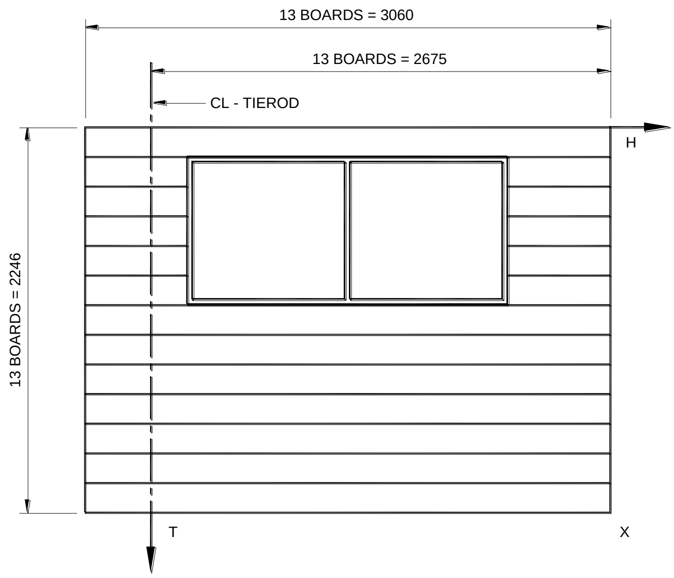
4/100mm x 4φ nails in shear	4 x 0.8 x 1.0 x 0.990	=	3.16 KN
4 Pryda Multigrips	4 x 0.8 x 7.6	=	24.32 KN
∴ Total Capacity		=	27.48 KN

which is more than adequate for the 14.508 KN maximum tension requirement.

(b) Check Concrete Floor Connection The Lockwood standard bracket incorporates two D10075 Dynabolts for fixing toa concretefloor. Assuming a minimum grade of concrete of 17.5 M.Pa and a 55mm anchor embedment, Ramset tests substantiate a capacity loading of 11.62 x 2 = 23.24KN which is adequate for the 14.508KN maximum tension requirement.

2. CASE 2 EXTERIOR WALLS

For external Lockwood walls the maximum tie rod tension required by an exterior wall bracing panel occurred with the 3.06m D2 winsow wall panel test carried out in 1986.



UPLIFT ON TIE RODS - EXTERIOR WALLS

From the test data the bracing unit capacity achieved was 272.0 B/U (Limit State).

$$H = \frac{272.0}{20}$$

By moments about point X

$$T \times 2.675 = H \times 2.246$$
$$= 13.60 \times 2.246$$

from which

T max = 11.419 KN

Substantial direct uplift forces on tie rods can also be induced by wind uplift during storm conditions. The worst uplift at exterior walls is experienced on trussed roofing when nett uplifts of the order of 4.0 KN per metre length occur under the $0.9G + W_u$ wind load condition.

From an inspection of the Lockwood standard plans the maximum length of exterior wall tied down by a single tie rod is 3.800m. Therefore the maximum (limit state) uplift than can be experienced by a single tie rod = 40 x 3.800 = 15.20 KN.

Check capacity of exterior tie rod fixing - 150 x 50 nog nailed through floor joists at each end with two 100mm x 4φ FH galvanised nails, 2 Pryda Multigrips from nog on to joists (12 nails per Multigrip - 3.15φ x 30mm FH Galv.), two Pryda 4N5 nail plates from nog to joists - detail B20 in the Lockwood Detail Manual.

<u>Capacity Loadings</u>

4/100mm x 4φ nails in shear	4 x 0.8 x 1.0 x 0.990	=	3.16 KN
2 Pryda Multigrips	2 x 0.8 x 7.6	=	12.160KN
∴	Total Capacity	=	19.248KN

which is more than adequate for the 14.508 KN maximum tension requirement.

3. TENSILE STRENGTH OF TIE RODS

From the preceding calculations, the maximum limit state tension requirement for a single tie rod is 15.20 KN. Lockwood standard tie rods are 9.0mm diameter galvanised with rolled threads, manufactured from Low Carbon Steel complying with Specification AISI 1010 with an ultimate strength of 385 M.Pa.

Assumed yield strength from AS 1250 (1981) $F_{yt} = 0.75 \times 385 = 288$ M.Pa.

Allowable yield strength in 9.0mm diameter bar $= \dfrac{288 \ x \ \pi \ x \ 9^2}{10^3 \ x \ 4} \ KN$

$=$ 18.32 KN which is adequate

SECTION E   TESTS ON LOCKWOOD COMPONENTS

Connections between adjacent wall sections are achieved by the "dove-tail" action of aluminium profiles fitting tightly into factory sawn grooves in the timber sections. There are many combinations of profiles and wall sections and these are well detailed in the Lockwood Assembly and Detail Manuals. There are, however, two basic profiles which are very important to the inherent stability of the Lockwood system - the X profile and the A or corner profile. These are shown full size on the accompanying diagram.

The integrity of the Lockwood system relies on these profiles tightly securing wall sections together. For example under wind suction on the lee side of a house the exterior wall must be adequately secured on to the adjacent walls or wall stiffeners to provide stability.

Tests have been conducted from time to time to ensure the adequacy of these joints.

1. X PROFILE TEST The attached diagram shows the basic test apparatus which was used to establish the ultimate strength of the X profile connection on to a single 172mm Lockwood board. The extension of the 62mm exterior board through to the clamp position proved necessary to stabilise the test assembly. In practice the boards are always restrained at each end, justifying the use of the end restraint. Tests were conducted in 1977, 1982, 1984 and 1991 with only minor variations to the test.

These are summarised below:-

	Date of Test	Dimensions mm L_1	L_2	L_3	Measured Load at Failure (Kg)	Calculated Load at X profile (Kg)
1	May 1977	880	984	224	585	523
2	May 1977	880	984	224	550	492
3	May 1977	880	984	224	690	617
4	June 1982	903	1013	152	850	757
5	June 1982	903	1013	152	900	820
6	June 1982	903	1013	152	1300	1158
7	Nov. 1984	820	950	100	1096	946
8	Nov. 1984	820	950	100	1165	1005
9	Nov. 1984	820	950	100	1180	1018
10	Nov. 1984	820	950	57	780	673
11	Nov. 1984	820	950	57	912	787
12	Nov. 1984	820	950	57	926	842
13	Nov. 1991	880	950	150	880	815
14	Nov. 1991	880	950	150	985	912

15	Nov. 1991	880	950	150	1011	936
16	Nov. 1991	880	950	150	1007	932
17	Nov. 1991	880	950	150	1046	968
18	Nov. 1991	880	950	150	1038	961
19	Nov. 1991	880	950	150	1169	1082
20	Nov. 1991	880	950	150	1057	979
21	Nov. 1991	880	950	150	1036	959
22	Nov. 1991	880	950	150	908	841
23	Nov. 1991	880	950	150	1078	1055
24	Nov. 1991	880	950	150	903	836

From NZS 4203 (1992) for Very High Wind Areas

($V_{d(z)}$ = 50 metres per second), the basic design wind pressure is

$q_{(z)}$ = $0.6\ V_{d(z)}^2 \times 10^{-10}$ KPa Eq 5.5.1

i.e. $q_{(z)}$ = $0.6 \times 50^2 \times 10^{-3}$

= 1.500 KPa

And for the worst wall suction condition C_{pi} = 0.2

and C_{pe} = -0.5 except for walls adjacent to exterior corners

From which the worst wall suction pressure = $1.5\ (C_{pe} - C_{pi})$

= 1.5 (-0.5 - 0.2) = -1.050 KPa

Hence the maximum wind (suction) loading that can be applied to an X profile connection, for stiffener or wall restraints at an assumed 4.000 metre maximum spacing in terms of NZS 4203 (1992) is:-

U = 1.050 x 0.172 x 4.0

= 0.722 KN per board (73.7 Kg per board)

From NZS 3603 (1993) Clause 10.6.2 the strength test load (TLB) shall be

TLB = $K_{30}\ K_{31}\ K_{32}\ \frac{U}{K_1}$ - Eq 10.2

K_1 = 1.0 for wind

K_{30} = 1.0 and K_{31} = 1.0 from tables

K_{32} = 1.0 (sample size > 19)

Whence TLB = 0.722 KN per board (73.7 Kg per board) Eq 10.2

The tests were carried out to failure to record the ultimate loading for each sample. Load deformation data was recorded for some of the samples. Refer to the attached graphs.

The mode of failure was found to vary, depending on the length of the wall extension past the X profile - shown as L3 on the diagram.

The shortest extension L3 in the Lockwood system is 57mm and this occurs at an interior wall-door jamb situation. More generally. L3 is significantly in excess of 150mm. Where L3 was greater than 150mm the mode of failure was generally by means of splitting along the grain of the bolted 43mm (vertical) member, as indicated on the diagram. In the samples with the extension L3 reduced to 57mm the failure mode was by means of splitting along the grain of the extension, as indicated.

The average Ultimate loading achieved at the X profile position from the test results was 873 Kg and the least Ultimate loading achieved on a single sample was 492 Kg. These achieved strengths are well in excess of the strengths required under Eq. 10.2, justifying the adequacy of the X profile connections.

2. CORNER (A TYPE) PROFILE TEST

The accompanying diagram shows the basic test apparatus which was used for tensile tests on the "A" profile corner in November 1984. Four samples were tested and typical graphical results are indicated. The test procedure adopted was very similar to the X profile test.

The test results are summarised below:-

Date of Test	Measure of Load at Failure (Kg)	Calculated Load at "V" profile
Nov. 1984	450	398
Nov. 1984	316	279
Nov. 1984	500	442
Nov. 1984	450	398

The mode of failure was by means of splitting along the grain of the horizontal member, away from the "V" profile as indicated on the diagram.

Analysing the data similarly to the X profile test.
From NZS 4203 (1992) for $V_{d(z)}$ = 50 metres per second again.

$q_{(z)}$ = 1.500 KPa

And for the worst wall suction condition C_{pi} = 0.2
C_{pe} = -0.65 (adjacent to wall corners)

From which the worst wall suction pressure = 1.5 (C_{pe} - C_{pi})

= 1.5 (-0.5 - 0.2) = -1.050 K.Pa

Hence the maximum wind (suction) loading that can be applied to an X profile connection, for stiffener or wall restraints at an assumed 4.000 metre maximum spacing in terms of NZS 4203 (1992) is:-

$$U = 1.050 \times 0.172 \times 4.0$$
$$= 0.722 \text{ KN per board (73.7 Kg per board)}$$

From NZS 3603 (1993) Clause 10.6.2 the strength test load (TLB) shall be

$$TLB = K_{30}\, K_{31}\, K_{32}\, \frac{U}{K_1} \quad - \quad \text{Eq 10.2}$$

K_1 = 1.0 for wind

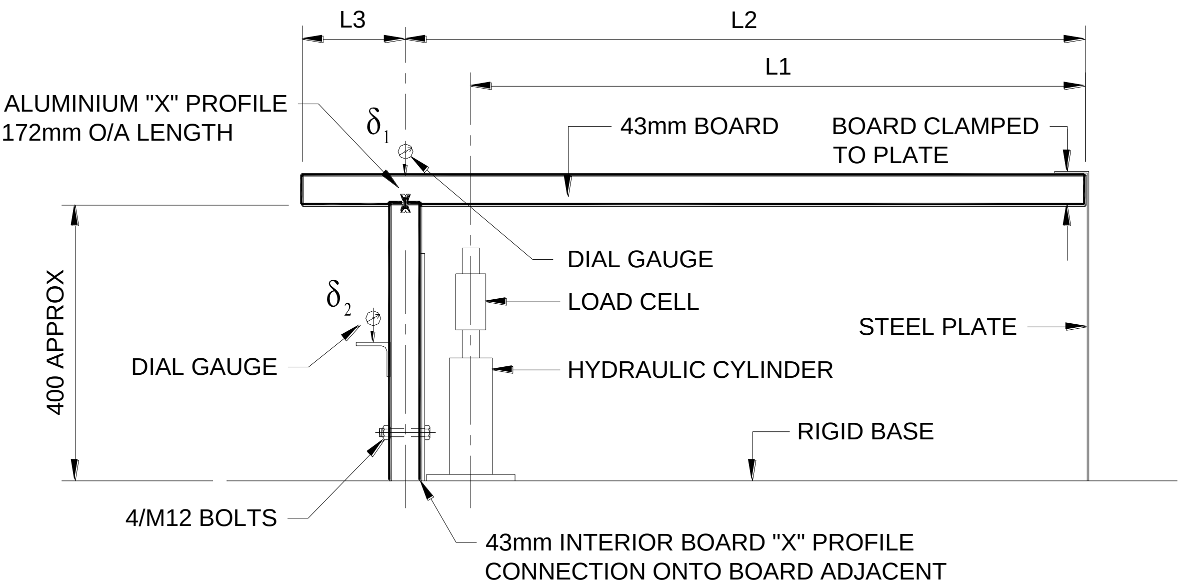
K_{30} = 1.0 and K_{31} = 1.0 from tables

K_{32} = 1.0 (sample size > 19)

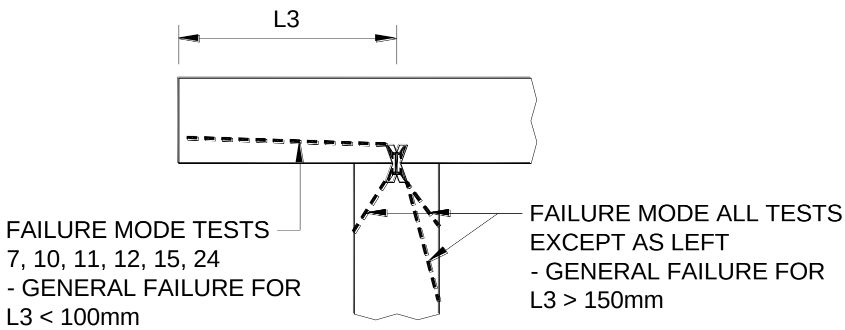
Whence TLB = 0.722 KN per board (73.7 Kg per board) Eq 10.2

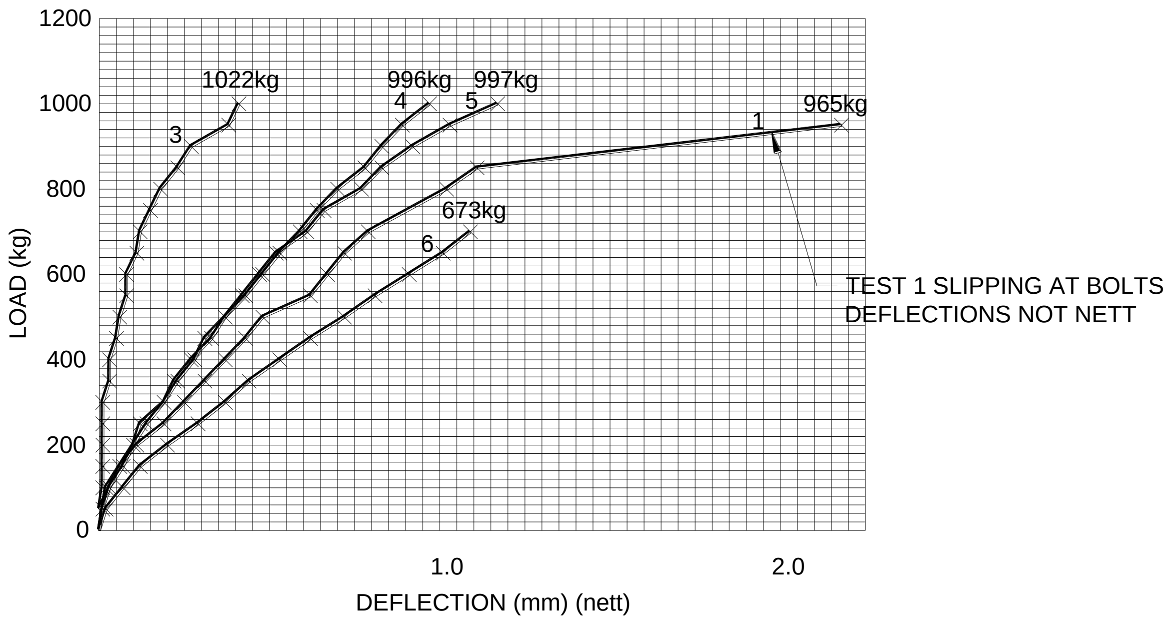
The average Ultimate loading achieved from the tests was 379 Kg and the least Ultimate loading achieved was 279 Kg.

These achieved strengths are well in excess of the strengths required under Eq 10.2, justifying the adequacy of corner (A type) profile connections.

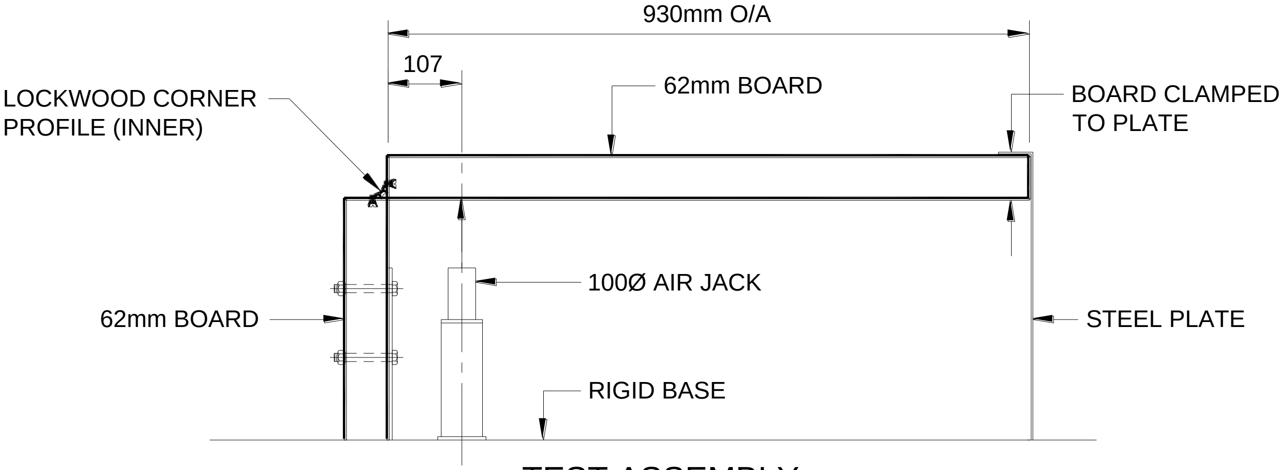


TEST ASSEMBLY

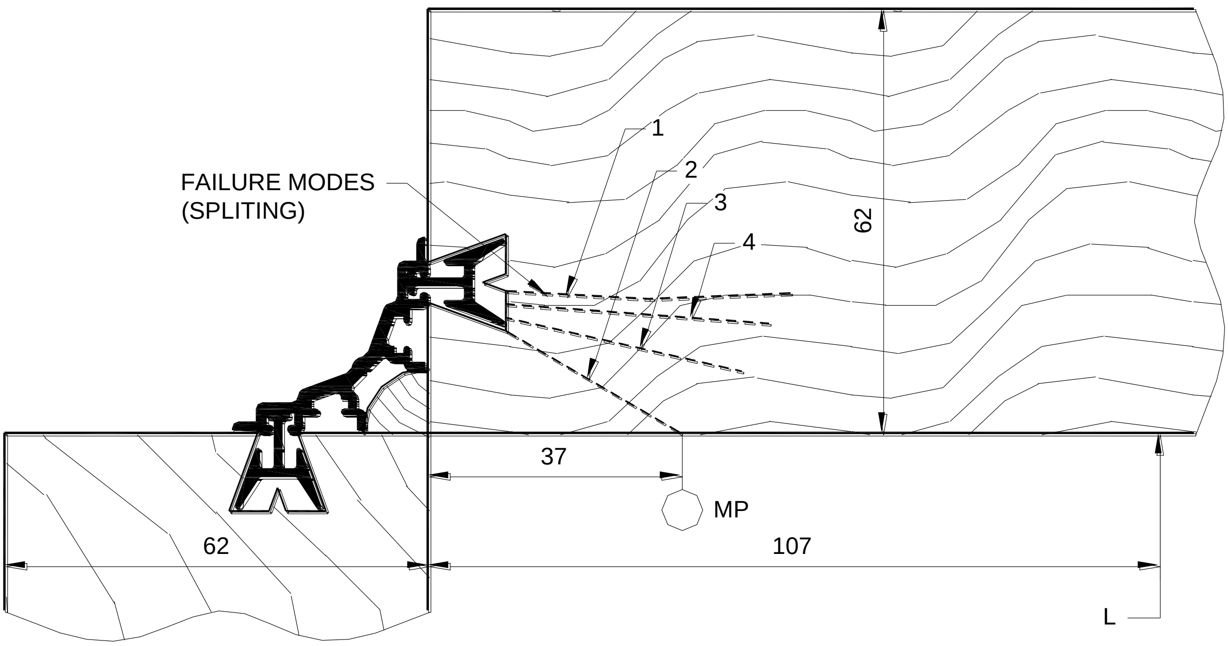


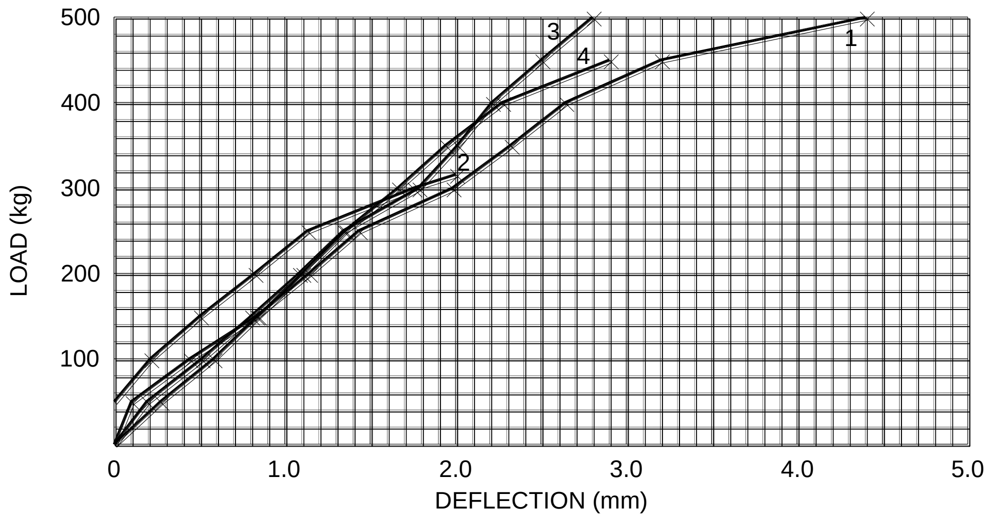


LOCKWOOD "X" PROFILE TEST - NOV 2000

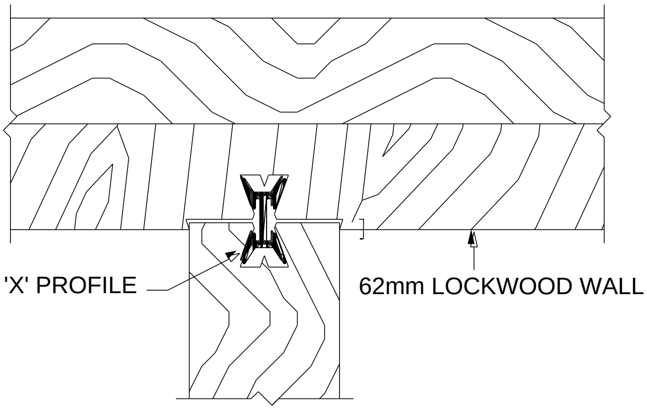


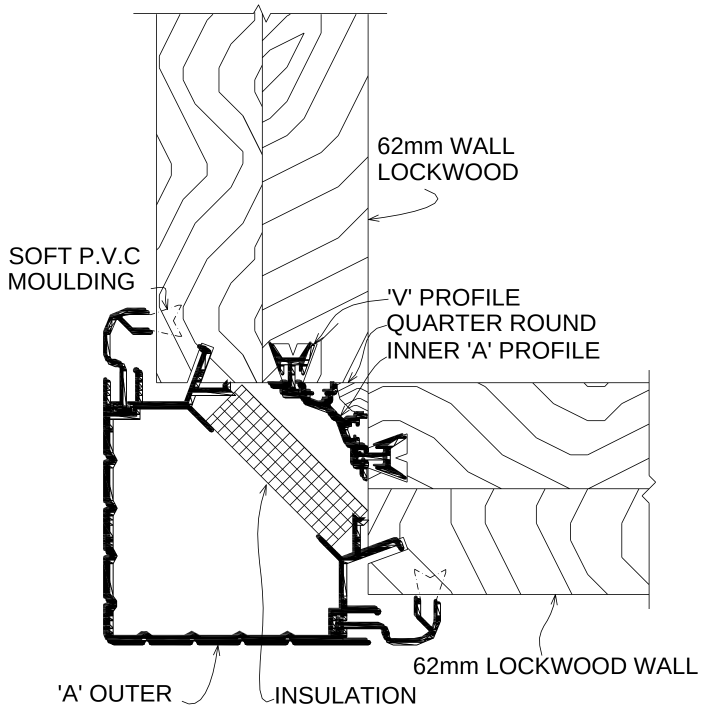
TEST ASSEMBLY





LOCKWOOD CORNER "A" PROFILE TEST - NOV 1984





LOCKWOOD `X' AND CORNER `A' PROFILE CONNECTIONS

SECTION F PANEL PRESSURE TESTS

1. SOLID EXTERIOR WALLS

Solid exterior walls (62mm) can be analysed from first principles, using code wind pressures and designing the Lockwood wall planks to span horizontally between return wall, or wall stiffener restraints. Bending stress in the 62mm wall planks is never a problem a limitation of deflection is the controlling criteria. Lockwood standard practice is to limit exterior wall panels to a maximum length of 4.500m.

2. EXTERIOR WALL WINDOW PANELS

Lockwood exterior wall panels incorporating doors and windows cannot be analysed for wind load conditions, and tests have been carried out on selected panels to substantiate their adequacy under wind load conditions.

The panels were selected for testing from the standard plans in the Lockwood Volume 2 Planbook. By inspection the critical panels selected for testing were: -

(i) N2 window panel 4.656m O/A Ref. Page F2

(ii) N3 window panel 4.800m O/A Ref. Page F3

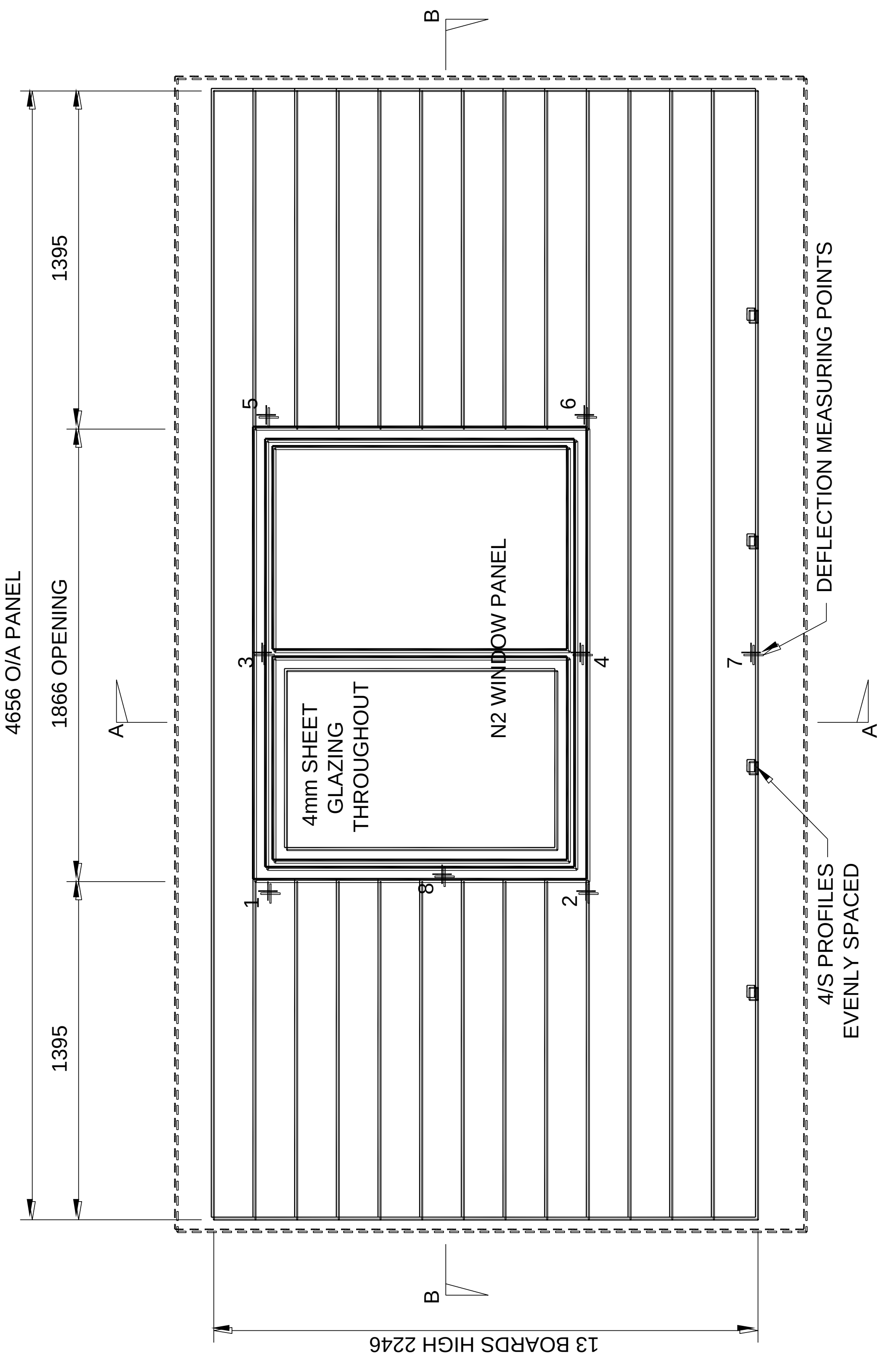
(iii) PS window panel 3.876m O/A Ref. Page F4

Each panel was pressure tested up to 3.514 KPa positive pressure and 3.034 KPa negative pressure – to substantiate the adequacy of the panels under extreme wind loading conditions. Each of the panels tested satisfactorily.

For the New Zealand Very High Wind Loading condition the test data was analysed in terms of NZS 3603 (1993) Ch. 10 “Testing of Timber Structures”. Each of the tested panels performed satisfactorily.

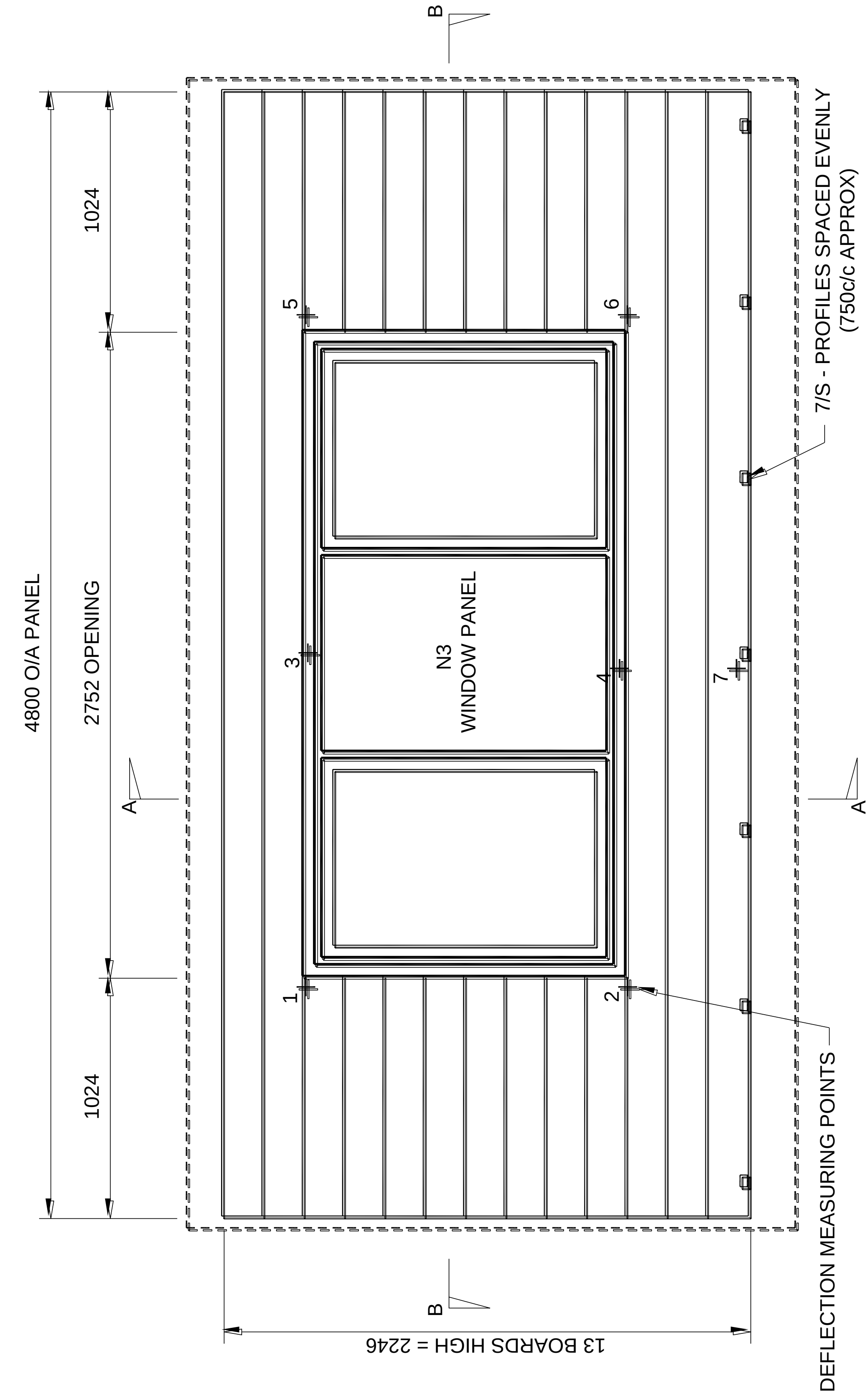
The tests on each of the three panels were identical. On pages F5-F16 is the pressure test report completed for the P.S. Window Panel.

LOCKWOOD N2 WINDOW PANEL - PRESSURE TEST



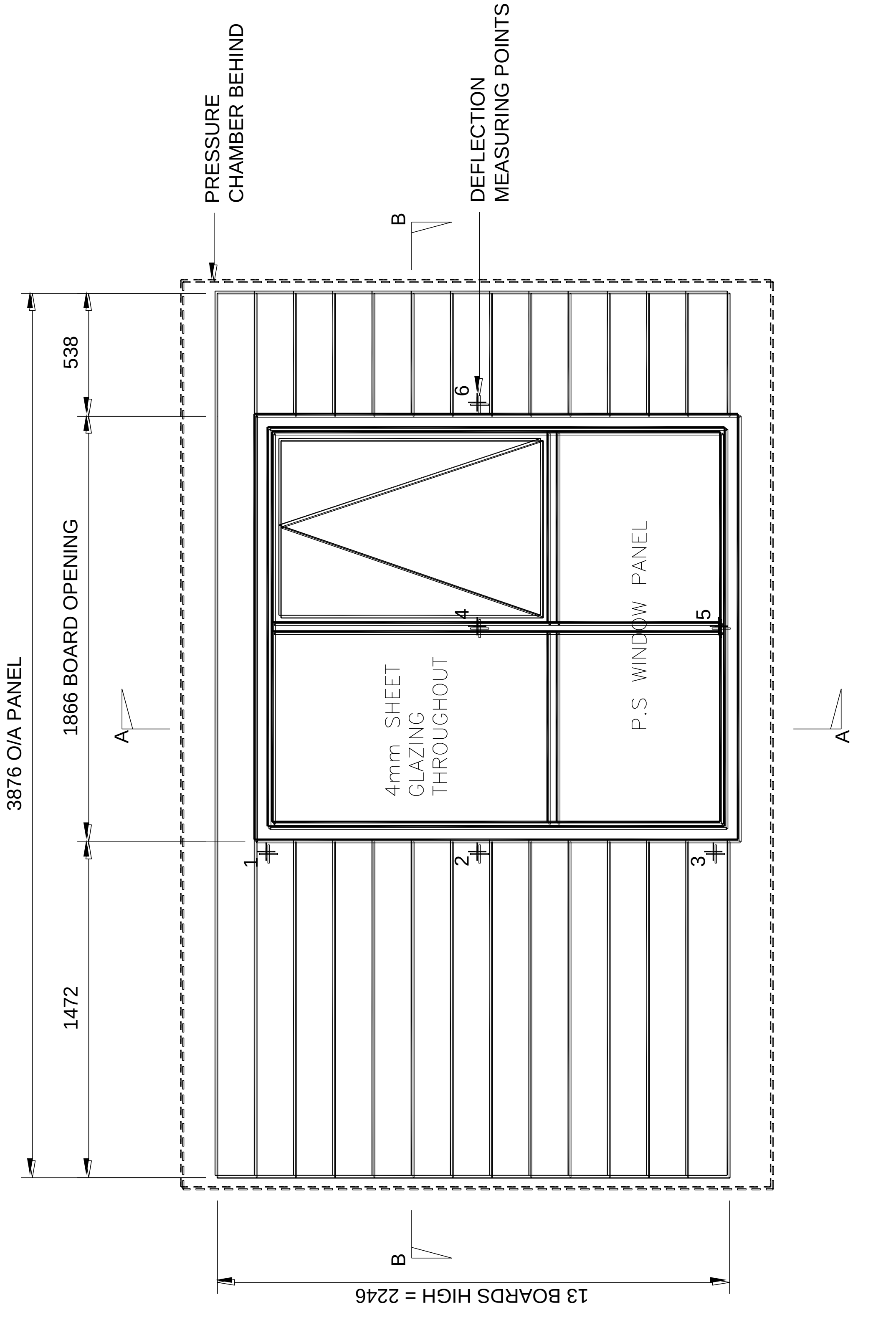
ELEVATION ON PRESSURE TEST PANEL 1:25

LOCKWOOD N3 WINDOW PANEL - PRESSURE TEST



ELEVATION ON PRESSURE TEST PANEL 1:25

LOCKWOOD P.S WINDOW PANEL - PRESSURE TEST



ELEVATION ON PRESSURE TEST PANEL 1:25

PRESSURE TEST

LOCKWOOD EXTERIOR WALL PS WINDOW PANEL

1. **GENERAL:** A pressure test was set up on a 3.876m Lockwood exterior wall, incorporating a standard P.S. Lockwood window joinery unit, on 9th and 10th May 1991 at the Lockwood laboratory in Russell Road, Rotorua (N.Z.).

   The test was set up, and procedures are in accordance with the requirements of NZS 3603 (1993) Chapter 10 "Testing of Timber Structures".

   The construction and testing of the wall panel was carried out under the supervision of Browne Spurr & Kronast, Consulting Engineers of Rotorua.

2. **TEST SPECIMEN:** The window panel was constructed as shown on the attached detail sheets, and in accordance with the Lockwood Assembly Manual and the Lockwood Detail Manual. The panel formed the front face of a pressure chamber with top and bottom fixings as shown in Section A-A - in line with standard details, and with end fixings nailed and screwed as Section B-B. Nailing and screwing was used at these positions to simplify construction of the test panel. In standard Lockwood construction the connections at ends of panels are continuously profiled into return walls or stiffening elements, and the adequacy of these profiled connections has been elsewhere fully substantiated by test.

   At the top of the panel the top board was fixed to the pressure chamber by nailing through the 100 x 50 timber plate as indicated on section A. The 100 x 50 plate was in turn screw fixed to the pressure chamber. The nailing was 100mm x 4φ F.H. galvanised nails through into the top of the top Lockwood board. This nailing is similar to the nailing incorporated in standard Lockwood construction between roof sarking and the top wall board.

   At the bottom of the panel a continuous S.J. aluminium seating profile was fixed to the bottom timber plate, screw fixed at 250mm centres through the centre of the profile. The bottom Lockwood board and the timber sub-frame cill of the aluminium P.S. joinery seated on to this profile, providing a mechanical connection to the (floor) pressure chamber.

   This continuous profile provided a stronger connection at floor level than is provided in standard Lockwood construction, where short lengths only of aluminium floor profiles are provided at 900mm maximum spacings under all exterior walls and full height joinery. A similar pressure test was subsequently set up for a N2 window panel, and in an extension of this test, the standard intermittent S profile fixings were incorporated. This successful test extension adequately substantiates the strength of the standard intermittent aluminium floor profiles.

   0.2mm black polythene sheeting was utilized on both sides of the wall panel to minimise air losses - this was necessary to assist with the pressure built up - particularly for the higher (cyclone) loading conditions.

3. **TEST APPARATUS:** The basic test apparatus is shown diagrammatically on the attached detail sheets. The chamber was pressurised by means of a two stage centrifugal tan, which could be used to provide positive or negative pressure by piping to the chamber from the outlet or inlet (respectively) of the fan. The

external face of the Lockwood wall panel faced into the pressure chamber. Positive pressurisation of the pressure chamber therefore produced a positive pressure on the wall - as indicated on the attached diagrams.

The pressure in the chamber was measured by means of a calibrated U tube manometer.

4. LOAD REQUIREMENTS: From NZS 4203 (1992) for Very High Wind Areas ($V_{d(z)}$ = 50 metres per second), the basic design wind pressure is

$$q_{(z)} = 0.6 \times V_{d(z}{}^2 \times 10^{-3} \text{ KPa} \quad - \text{ Eq 5.5. 1}$$

$$\text{i.e.} \quad q_{(z)} = 0.6 \times 50^2 \times 10^{-3}$$

$$= 1.500 \text{ KPa}$$

And for the worst wall (positive) pressure situation

$$C_{pe} = +0.7$$
$$C_{pi} = -0.3$$

From which the worst positive wall pressure

$$= 1.5\,(0.7) - (-0.3)$$
$$= 1.5 \times 1.0 = 1.5\text{KPa} = U$$

And for the worst wall negative pressure situation (suction)

$$C_{pe} = -0.5$$
$$C_{pi} = +0.2$$

From which the worst negative pressure (suction)

$$= 1.5\,(-0.5 - 0.2)$$
$$= -1.05 \text{ KPa (suction)} = U$$

<u>For the Stiffness Test Load (TLA)</u> For the Serviceability Limit State the Limit State Multiplier from NZS 4203 (1992) is 0.75 (0.93 for the Ultimate Limit State). From which the design wind speed for the Serviceability Limit State condition

$$= 50 \times \frac{0.75}{0.93} = 40.323\text{m/s}$$

And the corresponding design wind pressure q for the Serviceability Limit State

$$q = 0.6 \times 40.323^2$$
$$= 0.976 \text{ KPa.}$$

Again C_{pe} and C_{pi} = 1.000 for the worst positive wall pressure.
Therefore the pressure U for the Serviceability Limit State design condition

$$U = 1.0\,(0.976) = 0.976 \text{ KPa for positive pressure}$$

$$\text{and} \quad U = 0.7\,(0.976) = 0.683 \text{ KPa for negative pressure}$$

For the Strength Test Load (TLB)

$$TLB = K_{30}\, K_{31}\, K_{32}\, \frac{U}{K_1} \quad - \quad \text{Eq 10.2} \qquad \text{NZS 3603 (1993)}$$

K_1 = 1.0 for wind
K_{30} = 1.0 and K_{31} = 1.0 from tables
K_{32} = 1.66 for one sample tested and an assumed coefficient of variation of 0.20

$$\therefore \quad TLB = 1.66 \times \frac{1.5}{1.0} = 2.49 \text{ KPa for positive pressure}$$

And for the worst negative pressure (suction)

$$TLB = +1.66 \times 1.5\ (-0.5-0.2)$$
$$= -1.74 \quad \text{for negative pressure (suction)}$$

5. TEST PROCEDURE The chamber was pressurised and loading was applied in approximately 0.2 KPa increments up to ± 1.0 KPa and in 9.2 KPa increments up to ±2.0 KPa, with sets at zero pressure being measured at each 1.0 KPa (approx) increase in pressure. The panel was first pressurised in the positive pressure (windward) condition. Pressure was increased in increments up to 1.161 KPa and this pressure was held for 5 minutes. This pressure is in excess of the 0.976 KPa positive pressure requirement for the wall stiffness test load under maximum positive pressure for New Zealand wind conditions. The maximum deflections were recorded at point 4 - mid height on the central mullion of 16.42 and 16.52mm at the beginning and end (respectively) of the 5 minute loading period. Very similar deflections were also recorded at point 2 - mix height on the left hand jamb. After unloading the maximum deflection (set) recorded was 1.97mm.

The panel was then pressurised in the negative pressure (suction) condition in increments up to -1.055 KPa and this pressure held for 5 minutes. This pressure is in excess of the 0.683 KPa negative pressure requirement for New Zealand wind conditions. Again the maximum deflections were recorded at point 4, being 13.40 and 13.50mm at the beginning and end (respectively) of the 5 minute loading period. After unloading the maximum deflection (set) recorded was 0.36mm.

Note that the positive and negative loadings for the stiffness testing (to +1.161 and -1.055 KPa respectively) were carried out first - prior to the extension of the loadings for the strength testing. This offered the advantage of not unduly stressing and straining the panel before completing the two stiffness tests - where deflection is an important criteria.

The positive pressure test was then extended up to +3.514 KPa as a check on the strength of the panel. From reference to the graph, an irregularity is apparent between +1.161 and +1.200 KPa. This was caused by stressing panel to -1.932 KPa before continuing on with the positive pressure test above 1.161 KPa.

Finally the panel was then pressurised in the negative pressure (suction) condition to substantiate the adequacy of the strength of the panel under loading conditions. After maintaining the -3.034 KPa loading for three minutes, the top left hand (fixed) glazing panel shattered causing a sudden loss of pressure and the test was terminated.

The test results are tabulated on the attached sheets and the pressure - deflection characteristics are presented graphically.

6. ANALYSIS OF PRESSURE TEST RESULTS

(i) Stiffness Test Load The maximum recorded deflections under positive and negative loadings were +16.52mm and -13.50mm respectively) under the designated test pressure. After unloading the maximum deflections (sets) recorded were +1.97 and -0.36mm.

From the +1.161 KPa readings the factors RA at the six deflection points are:-

$$\frac{0.46}{5.77} = 0.080, \quad \frac{0.59}{14.35} = 0.041, \quad \frac{0.22}{2.15} = 0.102$$

$$\frac{1.97}{16.52} = 0.119, \quad \frac{0.52}{4.72} = 0.110, \quad \frac{0.08}{4.25} = 0.019$$

From the -1.056 KPa readings the factors RA at the six deflection points are:-

$$\frac{0.26}{5.16} = 0.050, \quad \frac{0.36}{13.37} = 0.027, \quad \frac{0.09}{1.60} = 0.056$$

$$\frac{0.36}{13.50} = 0.027, \quad \frac{0.06}{1.67} = 0.036, \quad \frac{0.15}{4.35} = 0.035$$

These figures are within the NZS 3603 (1993) suggested appropriate maximum value of

RA = 0.20 for this type of structure.

The maximum deflection measured in the stiffness test loading was in the positive (windward) loading to 1.161 kPa where a deflection of 16.52mm was recorded. By interpolation the deflection at 0.976 kPa was 13.81mm. With respect to the panel height of 2.246m this deflection represents $\frac{1}{162}$ of the height of the wall.

In our opinion this would be acceptable under extreme conditions (50 year storm).

(ii) Strength Test Load The criteria for acceptance under the Strength Test Load from NZS 3603 (1993) Clause 10.5.2 is "The unit shall pass the strength test if the test TLB is attained." TLB pressures of +2.49 kPa and -1.74 kPa were therefore required to be attained. In the test pressures of +3.514 kPa and -3.034 kPa were attained, justifying the strength of the panel.

7. FIXING OF ALUMINIUM JOINERY TO TIMBER SUB-FRAME

Lockwood aluminium joinery is completely prefabricated with a solid timber (ex 100 x 50) sub-frame around the full perimeter of the joinery unit. The timber sub-frame is connected to the Lockwood boards by aluminium X profiles at both jambs, by aluminium S.J. profiles at floor level, and by tongue and groove timber interlock at the heads and sills.

During an early test an inadequacy became apparent with respect to the manner in which the aluminium joinery was fixed to the timber sub-frame. Upon application of a high negative (suction) pressure the complete aluminium frame and glazing was sucked out of the timber sub-frame which remained firmly attached to the Lockwood boards. Upon examination of the joinery the cause of the failure was clear. Light gauge galvanised staples had been used to secure the aluminium frame around it's full perimeter to the timber sub-frame. A gap of approximately 5mm is maintained between the timber sub-frame, and the face of the aluminium frame at the fixing positions. The light gauge staples, under the negative pressure had bent over and pulled out of the timber. The use of staples in the joinery manufacture was a departure from earlier practice and a return to screw fixing is therefore strongly recommended.

The screw fixing procedure recommended for fixing aluminium joinery to timber sub-frames is to use 25mm x 6 gauge (3.4mm O/D threads) stainless steel posi-drive screws as below.

(a) To full height (2.1m high) joinery units, e.g. Z6, Z9, Z6P, PS, PSP, etc. Two screws each side of each full height mullion, top and bottom. Plus one screw at 500mm c/c maximum spacing around the full perimeter of the aluminium frame.

(b) To all other window joinery types N, H, J, D and E - one screw at 500mm c/c maximum spacing around the full perimeter of the aluminium frame.

All subsequent joinery used in the pressure tests, was fabricated using stainless steel screws in this manner, and no further problems were experienced.

8. GENERAL SUMMARY AND CONCLUSIONS

With reference to the Lockwood 3.876m wall panel incorporating a PS Lockwood joinery unit.

(i) In the pressure tests, involving positive and negative (suction) pressures, to substantiate the adequacy of the full panel and the joinery, the testing criteria set down in Chapter 10 of NZS 3603 (1993) was followed. For the highest New Zealand wind loading conditions the adequacy of the Lockwood wall panel and the P.S. joinery unit were fully substantiated by the test.

(ii) Fixing of Lockwood aluminium joinery to the timber sub-frame with stainless steel screws is strongly recommended. A test failure using the alternative method of stapling is detailed under Clause 7 of this report, together with recommendations for the screw fixing.

(iii) The breakage of the 4mm sheet glass in the upper fixed glazing panel under the application of the -3.034 kPa wind loading indicated a weakness which should be addressed. It is understood that 6mm sheet glass is used in all door sliding joinery as a safety feature in all Lockwood houses. 6mm sheet would offer more than twice the resistance to wind loading. It is also understood that 5mm sheet is now specified for use in "High Wind Kits" for New Zealand and overseas use. 5mm sheet would offer a 50% approximate increased resistance to wind loading and should be satisfactory, unless the glazing was struck by a flying object at the time of the storm.

J.K. SPURR
BROWNE SPURR & KRONAST

15/3/95

Structural handbook Amendment September 2013
Holmes Solutions
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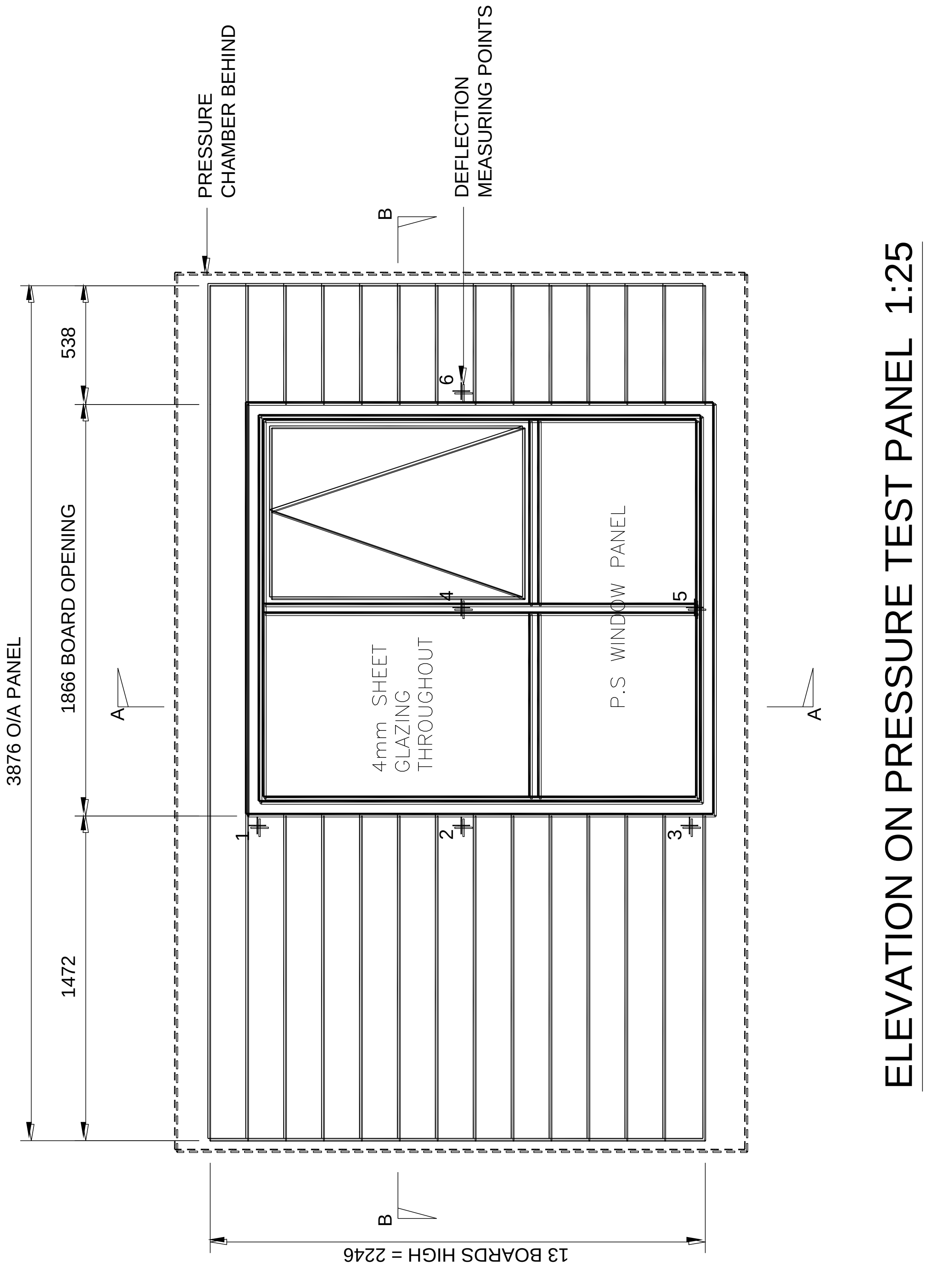
TEST RESULTS:
LOCKWOOD P.S WINDOW PANEL
PRESSURE TEST
DATE OF TEST – 9TH, 10TH MAY 1991
POSITIVE PRESSURE

PRESSURE (kPa)	**DEFLECTION ON FACE OF PANEL/JOINERY**					
	δ1	δ2	δ3	δ4	δ5	δ6
0.000	0	0	0	0	0	0
0.100	0.3	0.92	0.13	1.16	0.42	0.30
0.200	0.68	2.06	0.28	2.63	0.90	0.61
0.300	1.08	3.20	0.41	4.10	1.32	0.98
0.400	1.53	4.43	0.58	5.52	1.72	1.30
0.500	2.00	5.61	0.75	7.00	2.11	1.65
0.600	2.61	6.90	0.92	8.45	2.52	2.00
0.700	3.22	8.20	1.12	9.90	2.93	2.40
0.800	3.70	9.52	1.32	11.32	3.32	2.75
0.900	4.20	10.82	1.52	12.75	3.70	3.16
1.000	4.72	12.06	1.73	14.15	4.08	3.57
1.100	5.21	13.32	1.94	15.53	4.42	3.93
1.161	5.65	14.16	2.10	16.42	4.69	4.17
1.161 (5 minutes)	5.77	14.35	2.15	16.52	4.72	4.25
0.000	0.46	0.59	0.22	1.97	0.52	0.08
1.200	8.75	18.20	3.15	22.80	4.95	6.24
1.400	12.12	20.90	3.70	26.00	5.90	7.28
1.600	13.40	23.40	4.15	28.30	6.85	8.06
1.800	14.75	26.40	4.70	30.80	7.10	9.07
2.000	16.12	28.60	5.18	33.40	7.30	10.03
2.126	17.34	30.90	5.62	34.80	7.52	10.80
0.000	4.38	3.70	1.58	6.60	0.97	1.90
3.200	-	47.20	-	53.60	-	-
0.000	-	5.20	-	9.80	-	-
3.514	-	52.20	-	61.10	-	-
3.514 (5 minutes)	-	-	-	-	-	-
0.000	-	5.75	-	10.80	-	-

TEST RESULTS:
LOCKWOOD P.S WINDOW PANEL
PRESSURE TEST – NEGATIVE PRESSURE
DATE OF TEST – 9TH, 10TH MAY 1991

PRESSURE (kPa)	DEFLECTION ON FACE OF PANEL/JOINERY					
	$\delta 1$	$\delta 2$	$\delta 3$	$\delta 4$	$\delta 5$	$\delta 6$
0.0	0	0	0	0	0	0
-0.1	-0.38	-1.05	-0.13	-0.50	-0.16	-0.35
-0.2	-0.78	-2.15	-0.27	-1.60	-0.32	-0.70
-0.3	-1.28	-3.40	-0.40	-2.92	-0.50	-1.10
-0.4	-1.77	-4.70	-0.56	-4.55	-0.68	-1.48
-0.5	-2.27	-6.00	0.72	-5.94	-0.84	-1.90
-0.6	-2.78	-7.31	-0.88	-7.37	-1.02	-2.30
-0.7	-3.27	-8.56	-1.02	-8.72	-1.20	-2.75
-0.8	-3.78	-9.88	-1.18	-10.09	-1.32	-3.20
-0.9	-4.27	-11.20	-1.32	-11.37	-1.47	-3.60
-1.0	-4.74	-12.48	-1.48	-12.66	-1.60	-4.05
-1.055	-5.09	-13.24	-1.57	-13.40	-1.65	-4.30
-1.055 (5 mins)	-5.16	-13.37	-1.60	-13.50	-1.67	-4.35
0	-0.26	-0.36	-0.09	-0.36	-0.06	-0.15
-1.200	-5.40	-14.10	-1.70	-15.70	-1.74	-4.78
-1.400	-6.30	-16.80	-2.05	-18.00	-1.80	-5.75
-1.600	-7.16	-19.10	-2.37	-20.50	-1.82	-6.50
-1.800	-8.10	-22.00	-2.75	-23.50	-1.85	-7.50
-1.932	-8.66	-23.60	-2.90	-25.00	-1.90	-7.96
0	0.75	-1.00	-0.28	-1.00	-0.08	-0.65
-1.000	-5.50	-13.50	-	-17.00	-	-
-2.000	-11.00	-27.30	-	-28.50	-	-
-3.034	-15.00	-39.00	-	-41.00	-	-
1 Panel of Glass (4mm sheet) broke after load applied for 3 minutes						
0	-0.50	-1.20	-	-4.00	-	-

LOCKWOOD P.S WINDOW PANEL - PRESSURE TEST



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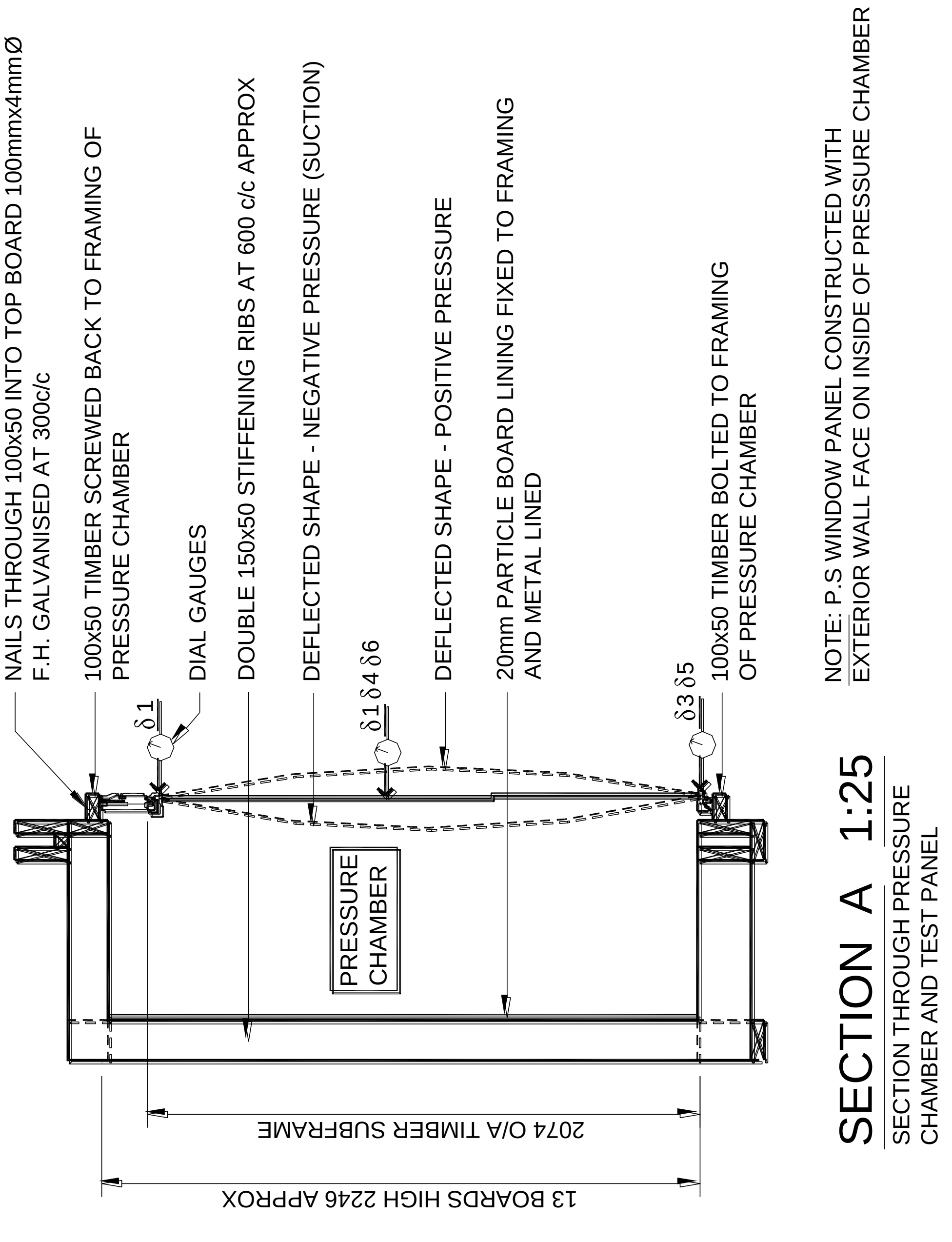

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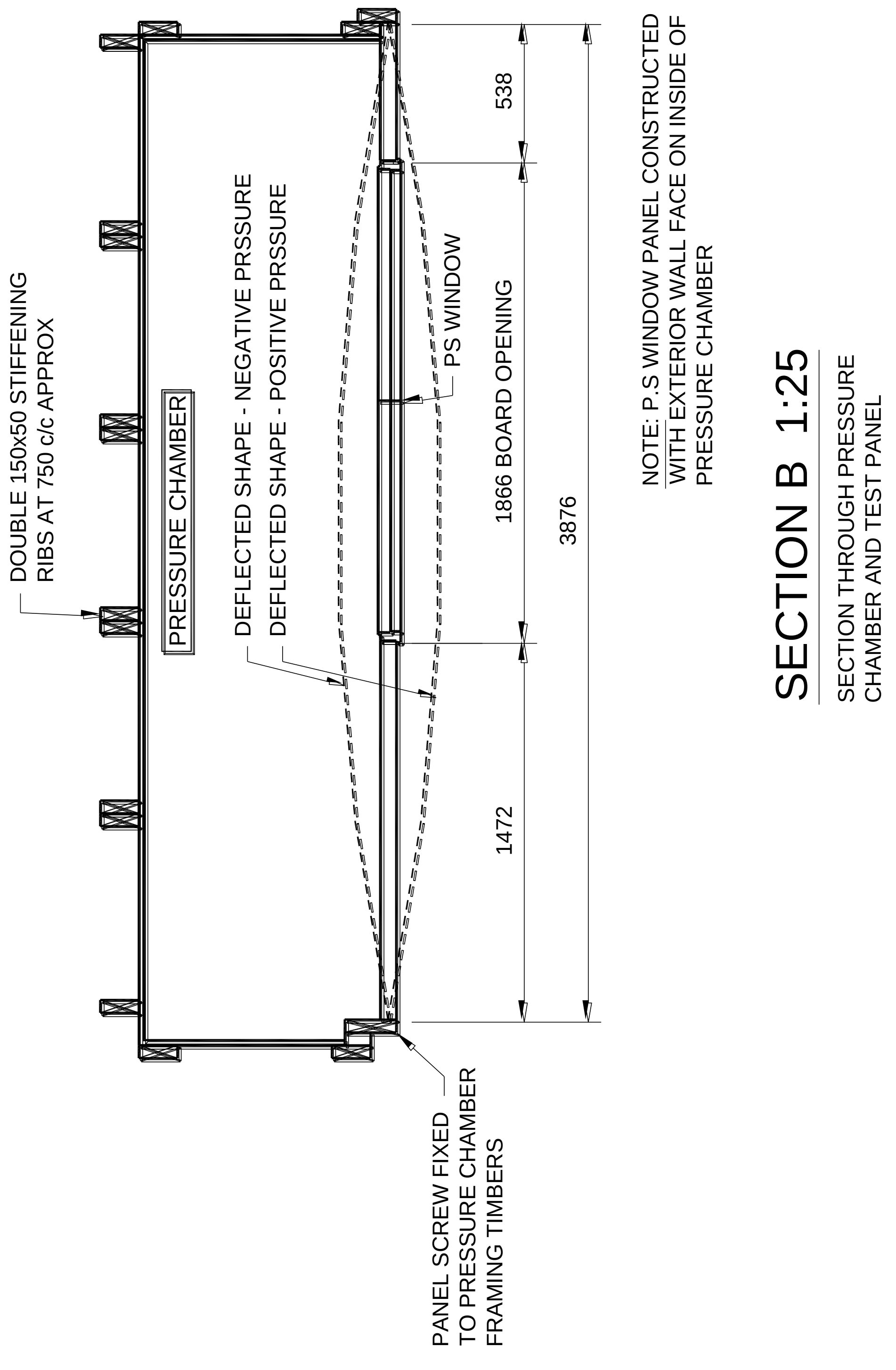

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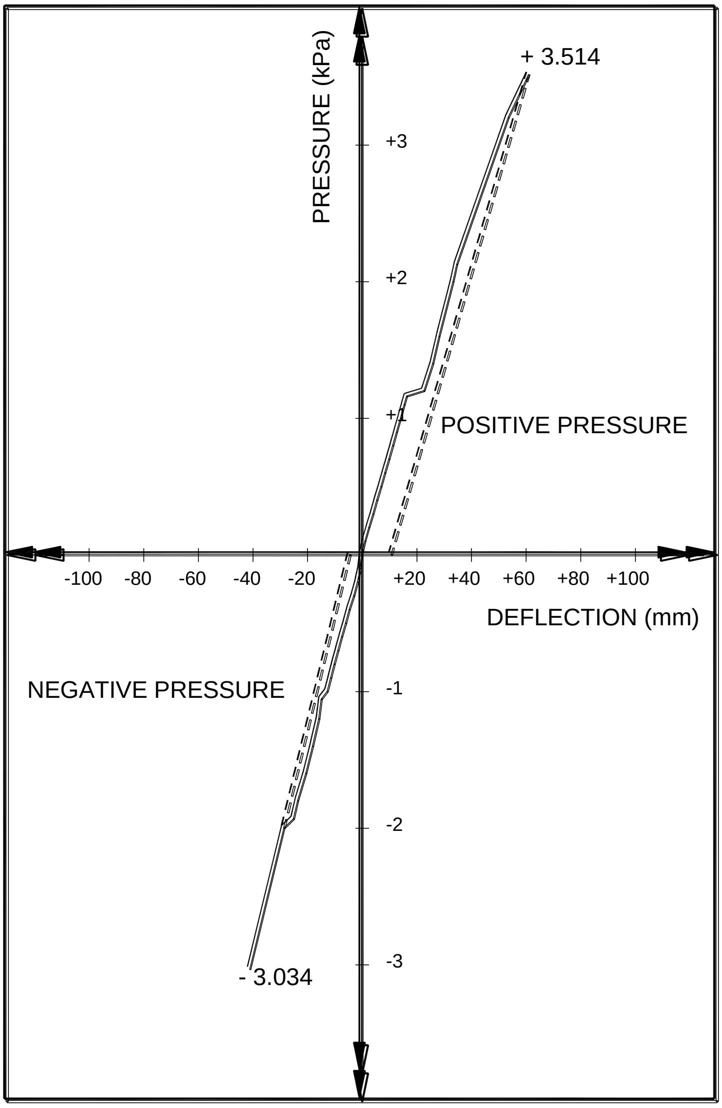

PRESSURE - DEFLECTION RELATIONSHIP AT POINT 4

3.876M - P.S. WINDOW PANEL