ENGINEERED BUILDING SOLUTIONS

Force 10 International Structural Design Calculations Australia and New Zealand

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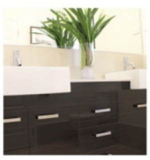

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Document Control

Revision	Date	Prepared By	Approved By
3.0	24 August 2011	Peter Lehrke	WJ Dalton
3.1	November 2012	Peter Lehrke – updates to Introduction P4 Wall Design Panel Criteria Table & W3 Cyclic Test Tables	WJ Dalton
3.2	February 2014	Peter Lehrke – updates to include NZBC requirements	WJ Dalton
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Ref: C:\DOCS\Force 10\Manuals\F10 05 STRUCTURAL DESIGN CALCULATIONS MANUAL VER 3.2.docx

1. INTRODUCTION

1.1 Purpose

The purpose of this Design Calculation Manual is to describe the **Force 10 Engineered Building System** to Designers, Engineers and approving Authorities. The information given in this manual should be sufficient as a reference and guide to the design of buildings up to an ultimate wind speed of C4 86 m/sec without additional information for compliance to wind loads as defined in the Australian NCC and New Building Codes.

Standard plans and layout are available for use however non-standard designs may be achieved by using the guidance and Tables in this Manual. All designs when submitted for approval to the local authority must meet all the relevant performance requirements of the relevant NCC or Australian, New Zealand (or International) Building Code.

The **Force 10 Engineered Building System** has been subjected to a through testing programme to ensure that the section capacities are correct and are in accordance with the Australian and New Zealand Standard Codes. The design information presented in the manual has been derived from engineering calculations and/or from testing.

This design calculation manual refers to building elements which are constructed in accordance with relevant building codes and standards. Where indicated, this manual must therefore be read in conjunction with that standard or the Australian NCC (BCA) or the New Zealand Building Code requirements.

Specific design will also be required for other elements such as strong backs, columns, beams, verandas etc. Specific design calculations will need to be prepared by appropriately qualified designers and a Design Certificate provided to cover each project.

1.2 Summary of System

The following information has been broken up into the various design elements of the system, ie. Floor, Wall, Roof, Components and gives a typical design example to suit cyclonic conditions in Australia.

Where testing has been used to define the design element, results have been calculated after taking into account the variability of the structural units.

Where calculations have been undertaken, the standard design code used AS/NZS4600 : 2005.

The following information will be sufficient for a design engineer competent in structural analysis to assess the system.

FORCE 10 BUILDING SYSTEM

Roof Design

Roofs have been structurally designed to current Australian and New Zealand Standard Codes over the lifetime of the Force 10 System.

Enclosed in this document is Design Calculations to AS/NZS4600: 2005 for an ultimate wind speed of 60m/sec and for an 11.0 module span. Computer analysis was undertaken using Space Gass 10.72(a) and results included. Design Analysis indicates that Force 10 members are satisfactory in combined loads.

(Ultimate uplift capacity from testing is 29.5 kN).

Wall Panel Design (Testing)

Testing was undertaken at the James Cook University Cyclone Testing Station in 2011. The test results were recorded and recommended Ultimate Limit State values were determined.

The series of tests were undertaken on the standard Force 10 Panels and for a panel developed for C4 Region D and the NCC Importance Level 4 (IPL4) in accordance with Standards Australia AS1170.2 – Wind Actions. This panel is referred to as the IPL4 panel and the same panel design is used as a C4 panel (all regions). The results are summarised as follows:

TEST REPORT	STANDARD PANEL	C4 PANEL	IPL4 PANEL
Bending Capacity KPa **Test Report -** TS826 (a) Non Cyclonic – Static Test	6.79 kPa	7.8 kPa	7.8 kPa
(b) Cyclonic – Cyclic Test	6.79 kPa	9.7 kN	9.7 kPa
Impact Test **Test Report -** TS829	-	-	Passed
Racking Resistance **Test Report** - TS830 (3 panels)	-	21.4 kPa	21.4 kPa
Tensile Strength **Test Report -** TS837	45.3 kPa	45.3 kPa	45.3 kPa

These values are ultimate section capacities and are to be reduced by the appropriate section capacities reduction factors of AS/NZS4600 – Table 1.6.

The test results were for 2435 mm high panels and have been extrapolated to 2700 mm high by the ratio squared – and similarly for 3000 mm high. Based on assessments of this testing, the following ultimate design capacities for the Force 10 panels can be summarized as follows:

DESIGN CAPACITY	STANDARD Up to C3	C4	IPL4
Racking Resistance (*kN*/Panel)			
(a) 2435 high	5.85	6.5	6.5
(b) 2700 high	5.27	5.7	5.7
(c) 3000 high	4.75	5.2	5.2

DESIGN CAPACITY	STANDARD Up to C3	C4	IPL4
Bending *kPa - Static*			
(a) 2435 high	6.79	8.74	8.74
(b) 2700 high	5.52	7.10	7.10
(c) 3000 high	4.47	5.76	5.76

In summary, Force 10 Panels can be used for all structures as nominated below for the Australian NCC:

Panel Type	Wind Classification	
	Class 1 & 10	Class 2-9
Standard Panel		
(a) 2435	N6 / C4	N6 / C3
(b) 2700	N6 / C3*	N5 / C2*
(c) 3000	N6 / C3	N5 / C2
C4 Design		
(a) 2435	N6 / C4	N6 / C4
(b) 2700	N6 / C4	N6 / C4
(c) 3000	N6 / C3	N5 / C2*
IPL4 Design		
(a) 2435	N6 / C4	N6 / C4
(b) 2700	N6 / C4	N6 / C4
(c) 3000	N6 / C3	N5 / C2*

* Updated November 2012

In summary, Force 10 Panels can be used for all structures as nominated below for the NZBC:

Panel Type	Wind Classification - Region	
	A6 and A7	W
Standard Panel		
(a) 2435	TC1	TC1
(b) 2700	TC1	TC1
(c) 3000	TC1	TC1
C4 Design		
(a) 2435	TC1	TC1
(b) 2700	TC1	TC1
(c) 3000	TC1	TC1
IPL4 Design		
(a) 2435	TC1	TC1
(b) 2700	TC1	TC1
(c) 3000	TC1	TC1

Panel Connections

Reference to Tensile Strength Test
– James Cook University Cyclone Testing Station Report TS837.

Tensile test was a panel assembly which is roof bolt – to panel – to floor bearer to stump using the Force 10 proprietary brackets. From this testing, the nominal tensile load (uplift) of the assembly is 45.3 kN. For a capacity reduction factor of 0.65 ⇒ tensile section capacity is 29.5 kN.

This value is characteristic of the system and requires no further review.

∴ **Section Uplift Capacity - 29.5kN**

Floor Design

Floors have been structurally designed to current standard codes over the lifetime of the structure.

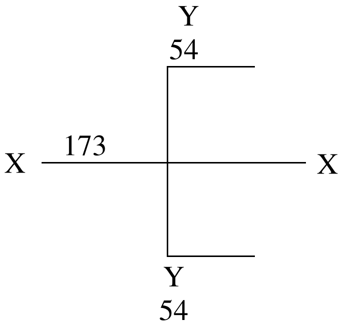
In the Floor Design area are Design Calculations to AS/NZS4600: 2005. From these calculations the Bearer / Joist Spans are calculated using 1.5 kPa Live Load, 2 kPa Live Load and 3 kPa Live Load.

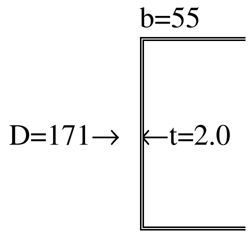
Footing Design

This is site specific and requires analysis of a Design Engineer and based on site specific knowledge.

FLOOR DESIGN

FLOOR BEARER – SECTION PROPERTIES





Idealised

$$I_y = 2\left(\frac{108^3 x2}{12}\right) = .42x10^6 mm^3 \quad r_y = 19.3, r_x = 65.5$$

$$I_x \; full \; section = 2\left(110x2\left(\frac{17}{2}\right)^3\right) + 2\left(\frac{169^3 x2}{12}\right) = 4.825x10^6 mm^4$$

$$Z_x = 55.78x10^3 mm^3 \quad A = 1124 \quad r_o = 68.3$$

$$I_w = \frac{a^2 I_y}{4} = 3.07x10^9 \quad J = W_F \, t^3/3 = 1500$$

Maximum Shear in Web - (Section 3.3.4)

$$\frac{d_1}{t} = \frac{171}{2} = 85.5$$

$$\sqrt{\frac{Ek_v}{f_y}} = \sqrt{\frac{2x10^5 x5.34}{550}}$$

$$= 44.07$$

$$\& 1.415\sqrt{\frac{Ek_v}{f_y}} = 62.35$$

$$\text{As} \frac{d_1}{t} > 62.35 \quad V_r = \frac{.905 E k_v t^3}{d}$$

$$= \frac{.905x2x10^5 x5.34x2^3}{171}$$

$$= 45.22kN \text{ Per Web}$$

$$\therefore \phi_v V_v = (2x45.22).9$$

$$\boxed{\phi_v V_v = 81.4kN}$$

Member Moment Capacity - (Section 3.3.3)

$$\lambda_b = \sqrt{\frac{M_y}{M_o}}$$

$$M_y = 450x55.78x10^3$$

$$= 25.1kNm$$

$$M_o = C_b A r_o \sqrt{f_{oy} f_{oz}}$$

$$C_b = 1.0 \quad A = 1124 \quad r_o = 68.3$$

$$f_{oy} = \frac{\pi^2 E}{\left(l/r_y \right)^2} \quad l = 500\text{ - Floor Joist Support}$$

$$= \frac{\pi^2 x2x10^5}{\left(\frac{500}{19} \right)^2 = 2940}$$

$$f_{oz} = \frac{GJ}{Ar_o^2}\left(1 + \frac{\pi^2 EI_w}{GJ\ell^2}\right)$$

$$= \frac{80000x1500}{1124x68.3}\left(1 + \frac{\pi^2 x2x10^5 x3.07x10^9}{80000x1500x500^2}\right)$$

$$= 4623$$

$$\therefore M_o = 1.0x1124x68.3\sqrt{4623x2940}$$

$$= 284kNm$$

$$\therefore \lambda_b = \sqrt{\frac{30.67}{284}} = .33$$

$$=> M_c = M_y$$

$$\therefore \phi_b M_c = .9x25.1$$

$$= 27.6kNm$$

$$\boxed{\phi_b M = 22.58kNm}$$

$\therefore$Clause 3.3.5 – Combined Bending / Shear

$$\boxed{\left(\frac{M^*}{22.58}\right)^2 + \left(\frac{V^*}{81.4}\right)^2 \leq 1.0}$$

Web Crippling – Section 3.3.6

$$R_b = Ct^2 f_y \sin\sigma\left(1 - C_r\sqrt{\frac{r}{t}}\right)\left(1 + C_1\sqrt{\frac{\ell_b}{t}}\right)\left(1 - C_w\sqrt{\frac{d}{t}}\right)$$

Using Table 3.3.6.2(B)

$C = 4,\quad C_r = .14,\quad C_1 = .35,\quad C_w = .02,\quad \phi_w = .85$

$r_i = 4.0,\quad t = 2.0,\quad \ell_b = 37,\quad d_1 = 171,\quad \sigma = 90^o$

$$\therefore R_b = 4x2^2 x550\left(1 - .14\sqrt{\frac{4}{2}}\right)\left(1 + .35\sqrt{\frac{37}{2}}\right)\left(1 - .02\sqrt{\frac{171}{2}}\right)$$

$= 8800(.8)(2.505)(.815)$

$= 14.37 kN / Web$

$R_b = 28.745$

$\therefore \phi R_b = .75x28.745$

$= 21.6$

$$\boxed{\phi R_b = 21.6kN}$$

$\therefore$Clause 3.3.7 – Combined Bending / Bearing

$$0.82\left(\frac{R^*}{21.6}\right) + \left(\frac{M^*}{22.58}\right) \le 1.32 \qquad (3.3.7(2)$$

OR $$\boxed{\frac{R^*}{34.77} + \frac{M^*}{29.81} \le 1.0}$$

Single Bearer Span – 3000 – 1.5 kPa LIVE LOAD

(1) Design Loads - Strength 1.2x.7+1.5x2 = 3.84 *kPa*
- Serviceability 0.7+.7x1.5 = 1.75 *kPa*

(2) Design Load Width – 2000 either side =

w_{ult} = 15.36*kN/m*
w_{ser} = 8.4*kN/m*

(3)

$$M^* = 15.36x\frac{9}{8}$$
$$= 17.28kNm$$
$$V^* = 15.36x\frac{3}{2}$$
$$= 23.0kN$$

Combined Bending / Shear Check

$$\left(\frac{17.28}{22.58}\right)^2 + \left(\frac{23}{81.4}\right)^2 = .66 < 1.0$$

Combined Bending / Bearing

$$= \frac{23}{34.77} + \frac{17.28}{29.81} = 1.24 > 1.0$$

To Achieve 1.0 Try $\frac{M^*}{29.81} = 1.0 - \frac{V^*}{34.77}$

$$M^* = 29.81 - .86V^*$$

ie. $\frac{w\ell^2}{8} = 29.81 - .86\frac{w\ell}{2}$

$$\ell^2 x\frac{15.36}{8} = 29.81 - .86x\frac{15.36}{2}\ell$$
$$= 1.92\ell^2 = 29.81 - 6.58\ell$$

OR $1.92\ell^2 + 6.58\ell - 29.81 = 0$

$$\ell = \frac{-6.58 \pm \sqrt{6.58^2 + 4x1.92x29.81}}{2x1.92}$$
$$= \frac{-6.58 + 16.5}{3.84}$$
$$= 2.58m$$

$\therefore$ Max Span for 2m Load Both Sides = 2.58m

(a) Deflection $\Delta = \frac{5w\ell^4}{384EI}$

$$\Delta = \frac{5x7.0x2740^4}{384x2x10^5x4.825x10^6}$$

$$= 5.3mm => \frac{Span}{510}$$

$$\left(For \Delta = \frac{Span}{600}\right)$$

$$\ell = 3\sqrt{\frac{384EI}{600x5x7}}$$

$$\ell = 2603$$

$\therefore$ For Joists either side 4000, Max Span = 2603

(b) Note : For Joists 3000 either side

$=> W_{sen} = 5.25kN/m$ and for $\frac{Span}{600}$

$$=> \ell = 3\sqrt{\frac{384EI}{600x5x5.25}}$$

= 2860 - Still less than 3 Mod

(c) Joists at 2000 either side

$$W = 3.5kN/m$$

$$= \ell = 3\sqrt{\frac{384EI}{600x5x3.5}}$$

$$= 3280 => 3\ Modules$$

(d) Joist 1500 either side

$$W = 1.75x1.5 = 2.625$$

$$= \ell = 3\sqrt{\frac{384EI}{600x5x2.625}}$$

$$= 3610$$

Summary				
Joist Spacing	4000	3000	2000	1500
Beam Span	2600	2860	3280	3610

However – based on product use since 1988, the recommended values can be rounded up without performance compromise

Performance Values				
Joist Spacing	4000	3000	2000	1500
Beam Span	2500	3000	3500	4000

Allowable spans 1.5 kPa LIVE LOAD

Single Bearer Span – 3000 – 2.0 kPa LIVE LOAD

(1) Design Loads - Strength $1.2x.7+2.0x2 = 4.84\ kPa$
- Serviceability $0.7+.7x2.0 = 2.1\ kPa$

(2) Design Load Width – 2000 either side =

$$w_{ult} = 19.36 kN/m$$
$$w_{ser} = 8.4 kN/m$$

(3)

$$M^* = 19.36x\frac{9}{8}$$
$$= 21.78kNm$$
$$V^* = 19.36x\frac{3}{2}$$
$$= 29.0kN$$

Combined Bending / Shear Check

$$\left(\frac{21.78}{22.58}\right)^2 + \left(\frac{29}{81.4}\right)^2 = 1.06 > 1.0$$

Combined Bending / Bearing

$$= \frac{32.67}{34.77} + \frac{21.78}{29.81} = 1.67 > 1.0$$

$$\text{To Achieve 1.0 Try } \frac{M^*}{29.81} = 1.0 - \frac{V^*}{34.77}$$
$$M^* = 29.81 - .86V^*$$
$$\text{ie. } \frac{w\ell^2}{8} = 29.81 - .86\frac{w\ell}{2}$$
$$\ell^2 x\frac{19.36}{8} = 29.81 - .86x\frac{19.36}{2}\ell$$
$$= 2.42\ell^2 = 29.81 - 8.32\ell$$
$$\text{OR } 2.42\ell^2 + 8.32\ell - 29.81 = 0$$
$$\ell = \frac{-8.32 \pm \sqrt{8.32^2 + 4x2.42x29.81}}{2x2.42}$$
$$= \frac{-8.32 + 18.915}{4.84}$$
$$= 2.18m$$
$$\therefore \text{Max Span for 2m Load Both Sides} = 2.18\text{m}$$

a. Deflection $\Delta = \dfrac{5w\ell^4}{384EI}$

$$\Delta = \frac{5x8.4x2740^4}{384x2x10^5 x4.825x10^6}$$

$$= 6.4mm => \frac{Span}{430}$$

$b. \therefore$ For Joists either side 4000, Max Span = 2380

Note : For Joists 3000 either side

$$=> W_{sen} = 6.3kN / m \text{ and for } \frac{Span}{600}$$

$$=> \ell = 3\sqrt{\frac{384EI}{600x5x6.3}}$$

= 2700 - Still less than 3 Mod

c.Joists at 2000 either side

$$W = 4.2kN / m$$

$$= \ell = 3\sqrt{\frac{384EI}{600x5x4.2}}$$

$$= 3092 => 3\ Modules$$

d. Joist 1500 either side

$$W = 2.1x1.5 = 3.15kN / m$$

$$= \ell = 3\sqrt{\frac{384EI}{600x5x3.15}}$$

= 3500

Summary				
Joist Spacing	4000	3000	2000	1500
Beam Span	2380	2700	3000	3500

However – based on product use since 1988, the recommended values can be rounded up without performance compromise

Performance Values				
Joist Spacing	4000	3000	2000	1500
Beam Span	2500	3000	3000	4000

Allowable spans 2.0 kPa LIVE LOAD

Single Bearer Span – 3000 – 3.0 kPa LIVE LOAD

(1) Design Loads - Strength 1.2x.7+2.0x3.0 = 5.34 *kPa*
- Serviceability 0.7+.7x3.0 = 2.8 *kPa*

(3) Design Load Width – 2000 either side =

w_{ult} = 21.36 *kN/m*
w_{ser} = 11.2 *kN/m*

(3) Try 2500 Span

$$M^* = 21.36x\frac{2.5^2}{8}$$
$$= 16.68kNm$$
$$V^* = 21.36x\frac{2.5}{2}$$
$$= 26.7kN$$

Combined Bending / Shear Check

$$\left(\frac{16.68}{22.58}\right)^2 + \left(\frac{26.7}{81.4}\right)^2 = 0.65 > 1.0$$

Combined Bending / Bearing

$$\frac{M*}{29.81} + \frac{V*}{34.77} \leq 1.0$$
$$= \frac{26.7}{34.77} + \frac{16.88}{29.81} = 1.33 > 1.0$$
$$\text{To Achieve 1.0 Try } \frac{M^*}{29.81} = 1.0 - \frac{V^*}{34.77}$$
$$M^* = 29.81 - .86V^*$$
$$\text{ie. } \frac{w\ell^2}{8} = 29.81 - .86\frac{w\ell}{2}$$
$$\ell^2 x\frac{21.36}{8} = 29.81 - .86x\frac{21.36}{2}\ell$$
$$= 2.67\ell^2 = 29.81 - 9.18\ell$$
$$\text{OR } 2.67\ell^2 + 9.18\ell - 29.81 = 0$$
$$\ell = \frac{-9.18 \pm \sqrt{9.18^2 + 4x2.67x29.81}}{2x2.267}$$
$$= \frac{-9.18 + 20.07}{5.34}$$
$$= 2.04m$$
$$\therefore \text{Max Span for 2m Load Both Sides} = 2.04\text{m}$$

(a) Deflection $\Delta = \dfrac{5w\ell^4}{384EI}$

$$\Delta = \frac{5x11.2x2000^4}{384x2x10^5x4.825x10^6}$$

$$= 2.4mm => \frac{Span}{800}$$

(b) For Joists 3000 either side

$$=> W_{sen} = 8.4kN/m \text{ and for } \frac{Span}{600}$$

$$=> \ell = 3\sqrt{\frac{384EI}{600x5x8.4}}$$

$$= 2450 \text{ - Still less than 3 Mod}$$

(c) Joists at 2000 either side

$$W = 5.6kN/m$$

$$= \ell = 3\sqrt{\frac{384EI}{600x5x5.6}}$$

$$= 2800 => 3\,Modules$$

(d) Joist 1500 either side

$$W = 4.2kN/m$$

$$= \ell = 3\sqrt{\frac{384EI}{600x5x4.2}}$$

$$= 3090$$

Summary				
Joist Spacing	4000	3000	2000	1500
Beam Span	2000	2450	2800	3090

However – based on product use since 1988, the recommended values can be rounded up without performance compromise

Performance Values				
Joist Spacing	4000	3000	2000	1500
Beam Span	2000	2500	3000	3000

Allowable spans 3.0 kPa LIVE LOAD

FLOOR JOISTS – G500

SECTION PROPERTIES – FULL SECTION

A=360.2	I_x=1.63x10^6	Z_x=34.32x10^3	r_x=67.3
J=187.6	I_y=.0764x10^6	Z_y=3.056x10^3	r_y=14.6

Maximum Shear – In Web – (Section 3.3.4)

$$\frac{d_1}{t} = \frac{95}{1.2} = 79.17, \sqrt{\frac{E_{kv}}{f_y}} = \sqrt{\frac{2x10^5 x5.34}{500}} = 46.22$$

$$1.415\sqrt{\frac{E_{kv}}{f_y}} = 65.40$$

$$\therefore \text{Where } {d_1}/{t} > 1.415\sqrt{\frac{E_{kv}}{f_y}}, V_v = \frac{.905Ek_v t^3}{d_1}$$

$$\therefore V_v = 17.58kN$$

$$\therefore \phi_v V_v = .9x17.58$$

$$= 15.823kN$$

$$\boxed{\phi_v V_v = 15.82kN}$$

Member Moment Capacity – (Section 3.3.3)

$$\lambda_b = \sqrt{\frac{M_y}{M_o}}$$

$$M_y = 500mPa \qquad M_o = \frac{\pi^2 EC_b dI_{xc}}{2\ell^2}$$

$$C_b = 1.0 \qquad I_{xc} = \frac{I_x}{2} \qquad d = 95 \qquad \ell = 3000$$

$$= \mathrm{M_o} = \frac{\pi^2 x2x10^5 x1.0x95x.815x10^6}{2x3000^2}$$

$$= 8.5kNm$$

$$M_y = 500x34.32x10^3 = 17.16$$

$$\lambda_b = \sqrt{\frac{17.16}{8.5}}$$

$$= 1.42 > 1.336$$ (Clause 3.3.3.2(4)

$$M_c = M_y\left[\frac{1}{\lambda^2{}_b}\right]$$

$$17.16x0.495$$

$$8.5kNm$$

$$\phi_b M_c = 0.9x8.5$$

$$= 7.65$$

$$\boxed{\phi_b M_o = 7.65kNm}$$

$$\therefore \text{Clause 3.3.5(1) - Combined Bending / Shear}$$

$$\left(\frac{\mathrm{M}^*}{7.65}\right)^2 + \left(\frac{V^*}{15.82}\right)^2 \leq 1.0$$

Spans

- **Design Loads**

(1) Dead Load = $.7kPa$ Live Load = $2.0kPa$

$$\therefore w = 1.2x.7 + 1.5x2$$

$$= 3.84kPa$$

$$= 3.84x.5kN/m$$

$$= 1.92kN/m$$

$$M^* = 1.92x\frac{3^2}{8} = 2.16kNm$$

$$V^* = 1.92x\,{}^3\!/_2 = 2.88kN$$

$$\therefore \left(\frac{M^*}{7.65}\right)^2 + \left(\frac{V^*}{15.82}\right)^2 = \left(\frac{2.16}{7.65}\right)^2 + \left(\frac{2.88}{15.82}\right)^2$$

$$= 0.11$$

(2) Serviceability

$$w = .7 + .7x2 \qquad (g + \Psi_s q)$$
$$= 2.1kPa$$
$$=> 1.05kN/m$$
$$\Delta = \frac{5w\ell^4}{384EI}$$
$$= \frac{5x1.05x3000^4}{384x2x10^5 x1.63x10^6}$$
$$= 3.397mm$$
$$ie. = \frac{Span}{880} < \frac{Span}{600} \quad \text{OK}$$

∴1.2mm JOIST OK TO 3000 SPAN

$$* \text{ NOTE FOR } \frac{Span}{600} \quad \text{ie.} \quad \frac{R}{600} = \frac{5\text{wl}^4}{384EI}$$

$$\ell = 5\sqrt{\frac{384EI}{600x5x1.05}}$$
$$= 3900 \quad \text{ie. Close enough to 4.0 mod}$$

DYNAMIC PERFORMANCE – LIGHT FLOOR SYSTEMS

AS3623 – APPENDIX D

Response to /kN Static Load - Section D2

Bearer Span - 3500
Load Width - 2500

CALCULATE P_d

$$k_b = \frac{E_b I_b}{\ell^3}$$

$$= \frac{2x10^5 x4.825x10^6}{3500^3}$$

$$= 22.51$$

$$k_c = \frac{E_c t_p{}^3 \ell}{12S^3}$$

$$= \frac{3000x19^3 x3500}{12x500^3}$$

$$= 48.0$$

$$\therefore \frac{k_b}{k_c} = 2.13$$

FIGURE D1 => $P_d = .8$

$$\therefore P_d = .8x1000 = 800N$$

$$\therefore \Delta = \frac{P\ell^3}{48EI}$$

$$= \frac{800x3500^3}{48x2x10^5 x4.825x10^6}$$

$$= .74mm > 1.0 \ \text{OK}$$

Unit Impulse – Section D3

NATURAL FREQUENCY

$$K_x = \frac{E_b I_b}{S}$$

$$= \frac{2x10^5 x4.825x10^6}{500}$$

$$= 1930x10^6$$

$$K_y = \frac{E_f t_r^{\ 3}}{12}$$
$$= \frac{3000x19^3}{12}$$
$$= 1.715x10^6$$
$$\therefore K_x >> K_y \quad \text{OK TO USE FORMULAE D3.2}$$
$$F = \frac{\pi}{2}\sqrt{\frac{K_x}{w\ell^4}}$$
$$DL \ \text{-} \ 30kg/m^2$$
$$LL \ \text{-} \ .3kPa \ \text{ie.} \ 30kg/m^2$$
$$= f = \frac{\pi}{2}\sqrt{\frac{1930x10^6 x10^{-3}}{60x3.5^4}}$$
$$= 23$$

Maximum Impact Velocity – Section D4

$$K_x >>> K_y \Rightarrow N_{40} = \frac{B}{L}\left(\frac{f^2-1}{r}\right)^{1/4}$$
$$B = 2.5 \quad L = 3.5 \quad F = \frac{40}{23} = 1.74$$
$$r = \frac{K_y}{K_x} = \frac{1.715}{1930} = .0009$$
$$N_{40} = \frac{2.5}{3.5}\left(\frac{1.74-1}{.0009}\right)^{1/4}$$
$$= 3.84$$
$$V_{\max} = 4\left(\frac{.4+.6x3.84}{60x3.5x2.5+200}\right)$$
$$= .0149m/\sec \quad (\text{or } 15mm/\text{sec})$$
$$\log_{10} V_{\max} < 1.2 + 2\sigma_o$$
$$2\sigma_o = 2\left(f_1 v \frac{.9}{100}\right) = 2\left(\frac{23x.9}{100}\right)$$
$$= 0.414$$
$$\therefore 1.2 + 2\sigma = 1.614$$
$$\log_{10} 15 = 1.176 < 1.614$$
$$\therefore \log_{10} V_m < 1.2 + 2\sigma$$

$\therefore$ FOR 2.5 WIDE FLOOR JOIST EACH SIDE BEARER SPAN

Check Bearing / Bending:

Live Load - 1.5*kPa* Dead Load - .7*kPa*

$$\therefore w = 1.5x1.5 + 1.2x.7$$
$$= 3.09kPa$$
$$w = 7.725kN/m \quad (2.5\text{ Wide})$$
$$Span = 3.5m \Rightarrow M^* = 7.725x\frac{3.5^2}{8}$$
$$= 11.83$$
$$R^* = 13.52$$
$$= \frac{R^*}{34.77} + \frac{M^*}{36.43} \leq 1.0$$
$$= \frac{13.52}{34.77} + \frac{11.83}{36.43}$$
$$= .71 < 1.0$$
$$\therefore \text{Bearer Acceptable}$$

WALL PANEL ANALYSIS

FORCE 10 WALL PANEL SYSTEM

Test Results - Wall Panel Assessment and Summary

(A) JCU CTS Cyclone Testing Station

Testing was undertaken in 2011 at the James Cook University Cyclone Testing Station. Testing was as follows:

TEST TITLE	REFERENCE
Wind Load Static Strength Test Of Standard Panel	**FORT 1101**
Static And Cyclic Wind Load Strength Testing Of IPL4 Wall Panels	**Report TS826**
Simulated Windborne Debris Impact Testing Of IPL4 Wall Panels	**Report TS829**
Serviceability, Static And Cyclic Racking Testing Of IPL4 Wall Panels	**Report TS830**
Static Tensile Strength Testing Of Wall Panel Stud System With Truss And Floor Connections	**Report TS837**

From these reports the following recommended ultimate strength design wind capacities were obtained.

Test Reference	Standard	IPL4
Bending Capacity (a) Static (b) Cyclic	 6.79 * -	 7.8 9.7
Impact test		Passed
Racking Resistance (3 Panels		21.4
Tensile Strength	45.3*	45.3*
	*Value determined by Author	

(B) BENDING CAPACITY (OUT OF PLANE BENDING)

From Testing:

Single Panel Test – Standard Panel

Load = 11.31 kPa

(1) STATIC TEST

Use 11.31 kPa as Value

For k_t = 1.5 => Ultimate Static 7.54 kPa

(2) CYCLIC TEST

Value = 0.9 x 11.31 kPa = 10.18 kPa

- ⇨ Test Load – if passes
- ⇨ Then Value = 9.16 kPa

These values are ultimate and need to have a capacity reduction factor applied => Ø = 0.9 => Value for 2435 High panels.

Non Cyclonic Ultimate Design Capacity (kPa)	Cyclonic Ultimate Design Capacity (kPa)
6.79	**6.79**
2435 panels	

To determine the bending capacity of other height panels by using previous testing results that have been undertaken since 1988 the bending capacity is equivalent to the inverse height ratio squared - therefore for other height panels:

Panel Height (H)	Ratio (2435/H)	Factor (Ratio x Ratio or R^2)	Ultimate Bending Capacity (kPa)	
			Standard	IPL4
2700	0.902	0.813	5.43	7.00
3000	0.812	0.659	4.44	5.56

A review of the Wind Load Codes (AS1170 and AS4055) gives the following net pressure for design. Comparing these values to the assessed capacities will give the following tabulated assessment.

Class	Wind Classification	Ultimate Wind Speed	Net Pressure Coefficient	Net Pressure
		V_n (M/Sec)	Cpn	P (kPa)
1-10	C3/N6	74	1.35	4.4
1-10	C4/N6	86	1.35	6.0
2-9	C2/N5	61	2.0	4.9
2-9	C3/N6	74	2.0	6.6

Note: Domestic structures (Class 1 and 10) can use Cpn of 1.35 where as Class 2 – 9 structures must use Cpn of 2.0.

Standard Panel:

Panel size	Wind Classification	
	Class 1 and 10	Classes 2- 9
2435	N6/C4	N6/C3
2700	N6/C3*	N5/C2*
3000	N6/C3	N5/C2

C4 /IPL4 Panel:

Panel size	Wind Classification	
	Class 1 and 10	Classes 2- 9
2435	N6/C4	N6/C4
2700	N6/C4	N6/C4
3000	N6/C3	N5/C2*

* Update November 2012

(C) RACKING TEST

From the James Cook University Cyclone Testing Station a recommended value for racking was set at 21.4 kN for 3 panels – normally 7.1 kN per panel.

It is recommended that the standard panel be further limited by bolt bearing loads.

Bearing capacity $ØV_b = Ø \propto C\, d_f\, t\, f_u =$ (AS/NZS 4600 Cl 5.3.4)
Where:

$Ø = 0.6$
$\propto = 1.0$
$d_f = 16mm$
$t = 1.2mm$
$f_u = 520Mk$

$C \Rightarrow \frac{d_f}{t} = \frac{16}{1.2} = 13.3$

$C = 4 - .1 \times 13.33$ Table 5.3.4 (B)
$= 2.667$

$\therefore ØV_b = 0.6 \times 2.667 \times 1.6 \times 1.2 \times 520$
$= \underline{15.98\ kN}$

For 2435 x 990 panel, from statics

Racking load x 2.435 = $ØV_b$ x 0.99 => Racking load 6.5 kN

For standard panel reduce by 0.9 => 6.5 kN x 0.9 = 5.85 kN

Ultimate Racking Value = 5.85 kN

Note – for higher panels racking value can be linearly extrapolated:

Recommended Ultimate Design Racking Loads		
Panel Height	Standard (kN /panel)	IPL4 (kN /panel)
2435	5.85	6.5
2700	5.27	5.7
3000	4.75	5.2

D AXIAL CAPACITY - LOADS – 2435 – 2700 HIGH PANELS

From previous testing on 2700 high panels (ASTM E 73 – 95) section 9 was determined as an appropriate test for the comparable load of a Force 10 wall panel.

(1) Axial Tests (RE-TEST Compression – CET4999/1)

Series	Test No.	Description	Results		Comments
			Failure	Allowable	
Retest CET – 4999/1	1	1 Panel	80.2	60.2	Failure. Panel continued to bend without increase in load. Local failure of studs at fixing brackets and sheeting.
	2	1 Panel	84.1	63.1	
	3	1 Panel	83.8	62.8	
Average				62.0	

Note: *(a) Retest – Compression Test on single panel to ASTM E72-95 Section 9*
(b) Allowable loads factored by 0.5

(2) Axial Tests (Compression & Tensile)

Series	Test No.	Description	Results		Comments
			Failure	Allowable	
3	1	1 Panel	30.1	22.58	Panel bolt rotating. Possible retest to simulate real site conditions.
	2	1 Panel	37.0	27.75	
	3	1 Panel	36.2	27.15	
Average				25.8	
4	1	1 Panel	28.2	21.15	Elongation of holes in studs - representative of real site conditions.
	2	1 Panel	37.7	28.28	
	3	1 Panel	36.4	27.30	
Average				25.58	

Note: *(a) Series 3 – Compression Test on single panel to ASTM E72-95 Section 9*
(b) Series 4 – Tensile Test on single panel to ASTM E72-95 Section 10
(c) Allowable loads factored by 0.75

The following recommended values have been determined by averaging test results.

Load Type	Nominal Member Capacity
Compressive	62.03*kN*
Tensile	25.58*kN*

E AXIAL LOADS – 3000 HIGH PANELS

Similar testing to the 2700 panels determined the following:

(1) Axial Tests (Compression & Tensile)

Series	Test No.	Description	Results		Comments
			Failure	Allowable	
6	1	1 Panel	78.0	58.5	Failure. Panel continued to bend without increased load. Local failure of studs at fixing brackets and sheeting
	2	1 Panel	74.7	56.03	
	3	1 Panel	75.0	56.25	
Average				57.0	
1 - 4	1	1 Panel	39.3	29.47	Elongation of holes in studs - representative of real site conditions
	2	1 Panel	44.4	33.3	
	3	1 Panel	44.7	33.5	
Average				32.1	

Note: (a) Series 6 - Compression Test on single panel to ASTM E72-95 Section 9
(b) Series 4 - Tensile Values Base on Racking.
(c) Allowable loads Factored by 0.75.

The following recommended values have been determined by averaging test results.

Load Type	Safe Working Load
Compressive	57.0 kN
Tensile	32.1 kN

F Axial Loads James Cook University Cyclone Testing Station

The James Cook University Cyclone Testing Station (Test Report T837) undertook a single test on a bolt assembly. From this test the breaking load was 55.9 kN. Applying a reduction factor of 0.81 the nominal tensile capacity of 45.3 kN is appropriate. The relevant capacity factor for the assembly is 0.65 giving the following:

Section Uplift Capacity = 29.5 kN

2435 Combined Bending & Compression AS/NZS4600 : P3.5

$$\frac{N^*}{\phi_c N_c} + \frac{C_{mx} M_x^*}{\phi_b M_{bx} \propto_{nx}} \leq 1.0 \qquad (3.5.1(1))$$

From Test Results

$$N_c = 62kN \quad M_{bx} = 6.79kPa$$
$$C_{mx} = 0.85 \quad \phi_C = .85 \quad \phi_b = .9$$
$$\propto_{nx} = 1 - \frac{N^*}{N_e}$$
$$N_e = \frac{\pi^2 EI_b}{(\ell_{eb})^2}$$
$$E \Rightarrow From\,Tests \quad P = 81kN \quad \Delta = 1.2mm \quad A = 2.6.1000mm^2 \quad I = 2700$$
$$E = \frac{P\ell}{A\Delta} = \frac{81x10^3 x2700}{12000x1.2}$$
$$= 15000MPa$$

$$I_b = 2\,Wall\ sheets\ separated\ 65mm$$
$$I_b = 2A_y^2 = 2x6000x\left(\frac{65}{2} + 3\right)^2$$
$$= 15.12x10^6 mm^4$$
$$\therefore \ell_{eb} = 2700mm$$
$$N_e = \frac{\pi^2 EI_b}{(\ell_{eb})^2} = \frac{\pi^2 x15000x15.12x10^6}{2700^2}$$
$$= 307.1kN$$
$$\therefore \propto_{nx} = \left(1 - \frac{N^*}{307}\right)$$
$$\therefore Eqn\,3.5.1(1)\,becomes$$
$$\frac{N^*}{.85x62} + \frac{.85M^*}{.9x6.79\left(1 - \frac{N^*}{307}\right)} \leq 1$$

OR

$$\boxed{\frac{N^*}{52.7} + \frac{M^*}{7.19\left(1 - \frac{N^*}{307}\right)} \leq 1.0}$$

2700 Combined Bending & Compression AS/NZS4600 : P3.5

$$\frac{N^*}{\phi_c N_c} + \frac{C_{mx} M_x^*}{\phi_b M_{bx} \propto_{nx}} \leq 1.0 \qquad (3.5.1(1))$$

From Test Results

$$N_c = 62kN \qquad M_{bx} = 5.43kPa$$

$$C_{mx} = 0.85 \qquad \phi_C = .85 \qquad \phi_b = .9$$

$$\propto_{nx} = 1 - \frac{N^*}{N_e}$$

$$N_e = \frac{\pi^2 EI_b}{(\ell_{eb})^2}$$

$$E \Rightarrow From\,Tests \qquad P = 81kN \qquad \Delta = 1.2mm \qquad A = 2.6.1000mm^2 \qquad I = 2700$$

$$E = \frac{P\ell}{A\Delta} = \frac{81x10^3 x2700}{12000x1.2}$$

$$= 15000MPa$$

$$I_b = 2\,Wall\ sheets\ separated\ 65mm$$

$$I_b = 2A_y^2 = 2x6000x\left(\frac{65}{2} + 3\right)^2$$

$$= 15.12x10^6 mm^4$$

$$\therefore \ell_{eb} = 2700mm$$

$$N_e = \frac{\pi^2 EI_b}{(\ell_{eb})^2} = \frac{\pi^2 x15000x15.12x10^6}{2700^2}$$

$$= 307.1kN$$

$$\therefore \propto_{nx} = \left(1 - \frac{N^*}{307}\right)$$

$$\therefore Eqn\,3.5.1(1)\,becomes$$

$$\frac{N^*}{.85x62} + \frac{.85M^*}{.9x5.43\left(1 - \frac{N^*}{307}\right)} \leq 1$$

OR

$$\boxed{\frac{N^*}{52.7} + \frac{M^*}{5.75\left(1 - \frac{N^*}{307}\right)} \leq 1.0}$$

3000 Combined Bending & Compression (AS/NZS 4600 : 3.5)

$$\frac{N^*}{\phi_c N_c} + \frac{C_{mx} M_x^*}{\phi_b M_{bx} \propto_{nx}} \leq 1 \qquad (3.5.1(1))$$

From Test Results

$$N_c = 57.0kN \quad M_{bx} = 4.44kPa$$

$$C_{mx} = 0.85 \quad \phi_c = .85 \quad \phi_b = .9$$

$$\propto_{nx} = 1 - \frac{N^*}{N_c}$$

$$N_c = \frac{\pi^2 EI_b}{(\ell_{eb})^2}$$

$$E = \text{FROM TESTS} \quad P = 68kN \quad \Delta = .99 \quad A = 2x6x1000 \quad \ell = 3000$$

$$E = \frac{P\ell}{A\Delta} = \frac{68000x3000}{12000x.99}$$

$$= 17171mm^3$$

$$I_b = 2Ay^2 = 2x6000x\left(\frac{65}{2} + 3\right)^2$$

$$= 15.12x10^6$$

$$\therefore Ne = \frac{\pi^2 x17171x15.12x10^6}{3000^2}$$

$$= 285kN$$

$$\therefore \propto_{nx} = \left(1 - \frac{N^*}{285}\right)$$

$$\Rightarrow Eqn\,3.5.1(1) \Rightarrow \frac{N^*}{.85x57} + \frac{.85M^*}{.9x4.44\left(1 - \frac{N^*}{285}\right)}$$

OR

$$\boxed{\frac{N^*}{48.5} + \frac{M^*}{4.70\left(1 - \frac{N^*}{285}\right)}}$$

ROOF DESIGN

SECTION PROPERTIES

Warping Constants - Chords

$$A \;=\; 280$$

$$Ix \;=\; 173.7\text{x}10^3$$

$$Iw = \frac{b^2 t}{6}\left(4c^3 - 6ac^2 + 3a^2 c + a^2 b\right) - m^2 Ixx$$

$$m = \frac{a^2 b^2 t}{Ix}\left(\frac{1}{4} + \frac{c}{2b} - \frac{2c^3}{3a^2 b}\right)$$

$$= \frac{40^2 \times 65^2 \times 1.2}{173.7 \times 10^3}\left(\frac{1}{4} + \frac{31}{2\times 65} - \frac{2\times 31^3}{3\times 40^2 \times 65}\right)$$

$$\therefore m = 13.9mm$$

$$Iw = \frac{65^2 \times 1.2}{6}\left(4\times 31^3 - 6\times 40\times 31^2 + 3\times 40^2 \times 31 + 40^2 \times 65\right) - (13.9^2)\times 173.7\times 10^3$$

$$= 845(119164 - 230640 + 148800 + 104000) - 33{,}560{,}587$$

$$\therefore Iw = 0.86\times 10^9$$

$$J = \frac{bt^3}{3} = \frac{(31+31+65+40)\times 1.2}{3}$$

$$\therefore J = 75$$

Warping Constants - Webs

$$m = \frac{36}{F+6} \qquad F = \frac{a}{b} = 1$$

$$= \frac{3\times 40}{7}$$

$$= 17.14$$

$$Iw = \frac{a^2 b^3 1.2}{12}\left(\frac{2+3}{1+6}\right)$$

$$= \frac{40^2 40^3 \times 1.2}{12}\left(\frac{5}{7}\right)$$

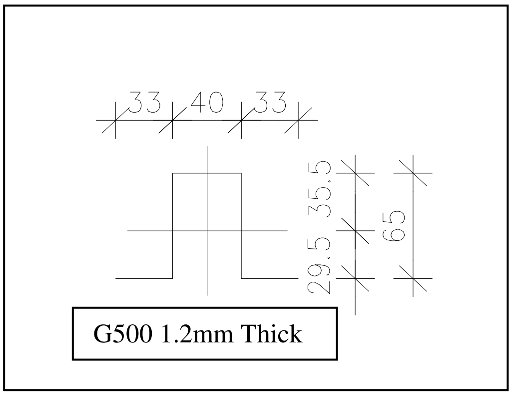
$$= 7.31\times 10^6$$

MEMBER CAPACITIES – ULTIMATE WIND SPEED 60m/s

Load Case 13 DL+WL 90 (One Truss)

Top Chord Worst Case

$\left(\ell = 2.129m \text{ - } Highest\ Axial\ Loads\right)$



Based on this orientation

Section Properties are :

$A = 280mm^2$ $\quad I_w = 8.4x10^7$

$I_y = 173.7x10^3$ $\quad I_x = 189.7x10^3$

$Z_y = 4.89x10^3$ $\quad Z_x = 3.579x10^3$

$r_y = 24.9$ $\quad r_x = 26.0$

$r_o = 38.7$ $\quad J = 75$

Nominal Member Moment Capacity (AS4600 Section 3.3.3)

$$F_c = {M_c}/{Z_f}$$

M_c depends on λ_b

$$\lambda_b = \sqrt{\frac{M_y}{M_o}}$$

$$M_y = F_y Z_y = 500x4.89x10^3$$

$$= 2.445x10^6\,kNm$$

$$M_o = C_b A r_o \left(\sqrt{F_{oy} F_{oz}}\right)$$

C_b can always be 1.0 $A = 280$ $r_o = 38.7$

$$F_{oy} = \frac{\Pi^2 E}{\left({\ell_y}/{r_y}\right)^2} \qquad \left(\ell_y = .75\ell = 1600\right)$$

$$= \frac{\Pi^2 E}{\left(\frac{1600}{24.9}\right)^2}$$

$$= 478MPa$$

$$F_{ox} = \frac{\Pi^2 E}{\left(\frac{1600}{26}\right)^2}$$
$$= 520$$
$$F_{oz} = \frac{GJ}{Ar_o^{\,2}}\left[1 + \frac{\Pi^2 EI_w}{GJ\ell^2}\right]$$
$$= \frac{80000x75}{280x1498}\left[1 + \frac{\Pi^2 x2x10^5 x8.56x10^7}{80000x75x1600^2}\right]$$
$$= 172MPa$$
$$\therefore M_o = 1.0x280x38.7\sqrt{172x478}$$
$$= 3.11kNm$$
$$\therefore \lambda_b = \sqrt{\frac{M_y}{M_o}}$$
$$= \sqrt{\frac{2.445}{3.11}}$$
$$= 0.887$$
$$\therefore M_c = 1.11M_y\left[1 - \frac{10\lambda^2{}_b}{36}\right] \quad -3.3.32(4)$$
$$= 1.11x2.445\left[1 - \frac{7.86}{36}\right]$$
$$= 2.12kNm$$
$$\therefore f_c = \frac{2.12x10^6}{4.89x10^3}$$
$$= 434MPa$$
$$\therefore M_o = Z_c F_c$$
$$= 434x4.89x10^3$$
$$M_b = 2.12kNm$$
$$\therefore \phi_b M_b = .9x2.12$$
$$= 1.91kNm$$

BEND PER METRE

$$\boxed{\phi_b M_b = 1.91kNm}$$

Nominal Member Compression Capacity 3.4.3

$$\lambda c = \sqrt{\frac{r_y}{F_{oc}}}$$

$$F_y = 550$$

$$F_{oc} = \frac{1}{2\beta}\left[F_{ox} + F_{oz} - \sqrt{(F_{ox} + F_{oz})^2 - 4\beta F_{ox} F_{oz}}\right]$$

$$\beta = 1 + \left(\frac{r_x}{r_o}\right)^2 \quad F_{ox} = 520$$

$$= 1.45 \qquad F_{oz} = 172$$

$$= .3445\left[(520 + 172) - \sqrt{(520 + 172)^2 - 4x1.45x520x172}\right]$$

$$= .3445[692 - 199]$$

$$= 170MPa$$

$$\therefore \lambda_2 = \sqrt{\frac{500}{170}}$$

$$= 1.71$$

$$F_a = \left(\frac{.877}{1.71^2}\right)500$$

$$= 150MPa$$

$$\therefore N_c = 150x280$$

$$= 42$$

$$\phi_c N_c = .85x42$$

$$= 35.7$$

$$\boxed{\therefore \phi_c N_c = 35.7kN}$$

Nominal Section Capacity - Tension (3.2.1)

$$N^* = \phi_t N_T$$
$$\phi_t = .9$$
$$N_T = A_y F_y$$
$$= 280x500 = 140kN$$
$$or \quad .85k_T A_n F_N$$
$$k_T = 1.0 \quad A_n = 280 \quad F_u = F_y = 550$$
$$\therefore N_T = .85x280x500$$
$$= 119kN$$
$$\therefore N^* = .9x119$$
$$= 107kN$$
$$\boxed{\phi_t N_T = 107kN}$$

SUMMARY

$$\phi_b M_b = 1.91kNm$$

$$\phi_c N_c = 35.7kN$$

$$\phi_t N_t = 107kN$$

TRUSS MEMBERS - Combined

$$\frac{N*}{\phi_c N_c} + \frac{C_{mx} M*}{\phi_b M_b \alpha_{nx}} \leq 1.0$$

$$C_m = .6 - .4\left(\frac{M_1}{M_2}\right)$$

$$M_1 = .35, M_2 = .75 - \text{Computer Case 1}$$

$$= .6 + .4\left(\frac{.35}{.75}\right) = .79 \ \text{(Double Curvature)}$$

$$\alpha_{\text{nx}} = 1 - \frac{N_x}{N_e}$$

$$N_e = \frac{\Pi^2 EI_y}{(\ell)^2}$$

$$= \frac{\Pi^2 Ex173.7x10^3}{1600^2}$$

$$= 134kN$$

$$\therefore \frac{N*}{\phi_b M_o \left(1 - \frac{N*}{134}\right)}$$

$$\therefore \frac{N^*}{34} + \frac{.79M^*}{2.2\left(1 + \frac{N^*}{134}\right)} \leq 1.0$$

General Formula

$$\boxed{\frac{N*}{34} + \frac{M*}{2.42\left(1 - \frac{N*}{134}\right)} \leq 1.0}$$

REVIEW OF COMPUTER RESULTS

Wind Load Cases

Top chords in tension - Max $N^*=48kN$
Max $M^*=.75kNm$

Bottom chord compress - Max $N^*=7.5kN$
Max $M^*=.53kN$

Neither approach capacity

Webs - Max $N_c^*=9.42kN$
(Member 21 – Case 13)

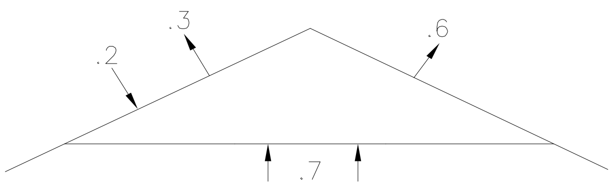
(Length = 1.77m with fixity) $\therefore$ Safe to use 1.6 calculations

$\therefore \phi N_c=34kN$

Reactions

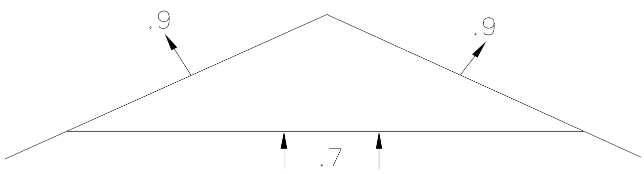
Uplift = $19.1kN$

Loads For Computer "C2" or "TC1"Wind Area



WL 0°
WL 90° (one truss)

Trusses 1000 CTS



Dead Load

$1.0kPa$ on Roof - $.10kN/m\ (ult = .15)$

$1.0kPa$ on Bottom Chord $= .10kN/m\ (ult = .15)$

$Wind\ = 60m/\sec => q_u = 2.16kPa$

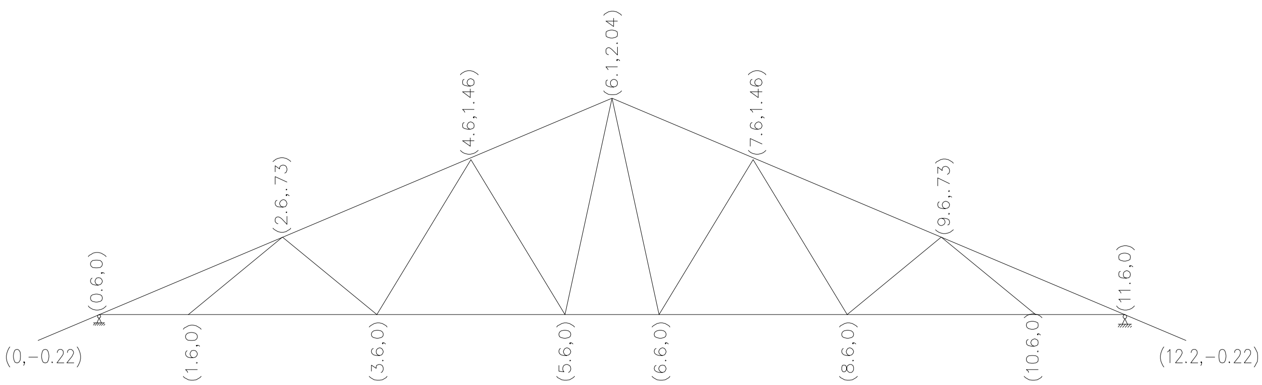
Live Load

$$.12 + \frac{1.8}{11} = .29\,kPa\ (ULT\ = .44\,kPa\ )$$

—

<u>Wind Loads</u> (1000CTS)

WL0°	=>	-.3x2.16	=	-.648kN/m
		-.6x2.16	=	-1.296
		+.7x2.16	=	+1.512
WL90°	=>	-.9x2.16	=	-1.944
		+.7x2.16	=	+1.512



ALL MEMBERS ⎍ 1.2 G550

COMPONENT FIXINGS

F10 COMPONENT CHECK

A. Top Bracket

40mm x 8 Plate, M16 Stud, 40-24 Weld. (6mm CFW)

(1) PLATE – Tensile Capacity (AS4100)

Clause 7.1

$$N^* \le \phi A_g f_y \text{ OR } (.85 k_t A_N f_u)\phi$$
$$A_y = 40x8 = 320 \quad f_y = 300 \quad k_t = 1.0 \quad A_n = (40-17)x8 = 184$$
$$f_u = 400 \quad \phi = .9 \quad k_t = 1.0$$
$$N^* = .9x320x300 \text{ or } (.85x184x400).9$$
$$= 86.4 \text{ or } 56.3kN ----------*$$

(2) M16 STUD - Clause 9.3 (AS4100)

Clause 9.3.2.2

$$N_{tf}^x = \phi N_{TF}$$
$$\phi = .8 \quad f_{uf} = 400 \quad A_s = 157$$
$$\Rightarrow N_{tf}^x = .8x400x157$$
$$= 50.24kN ----------*$$

(3) WELD – Clause 9.7 (AS4100)

$$V^*{}_w = \phi V_w$$
$$V_w = .6 f_{uw} t_t k_r$$
$$f_{uw} = 410 \quad t_t = 4.24 \quad k_r = 1.0 \quad \phi = .6$$
$$= V_w^* = (.6x410x4.24).6$$
$$= .626kN/mm$$
$$\ell = 32mm \Rightarrow V_w^* = 20.03kN ----------*$$

$$\therefore \quad \boxed{\text{Top Bracket } \phi N_t^* = 20kN}$$

B. Stud Material

G500 material 1.2mm thick

(A) SECTION PROPERTIES

Assume sheeting act as stiffened lip.

$$Flange, {}^{b}\!/_{t} = \frac{36-6}{1.2} = 25.0$$

$$Web, {}^{b}\!/_{t} = \frac{64-12}{1.2} = 43.3$$

Effective widths – (AS4600 Clause 2.2.1.2)

$$\lambda = \sqrt{\frac{f_x}{f_{cr}}}$$

$$f_{CR} = \left(\frac{k\pi^2 E}{12(1-v^2)}\left(\frac{t}{b}\right)\right)^2$$

(i) WEB $k = 4.0 \quad t = 1.2 \quad b = 36 \quad v = .3$

$$f_{cr} = \frac{4.x.10^5 x\pi^2}{12(1-.3^2)}\left(\frac{1.2}{36}\right)^2$$

$$= 850$$

$$\therefore \lambda = \sqrt{\frac{500}{850}}$$

$$= .767$$

$$\rho = \frac{\left(1-\frac{.22}{..767}\right)}{.767}$$

$$= .93$$

$$\therefore b_{ef} = \rho b$$

$$= 33.48$$

(ii) FLANGE $k = 4.0 \quad t = 1.2 \quad b = 52 \quad v = .3$

$$f_{cr} = 384$$

$$\lambda = \sqrt{\frac{500}{384}} = 1.20$$

$$\Rightarrow \rho = \frac{1-\frac{.22}{1.20}}{1.20} = .672$$

$$\therefore b_{ef} = .672x52$$

$$= 35$$

$$\text{(iii) AREAS - GROSS} \approx (36+36+64)1.2 =$$
$$A_G = 163mm^2$$
$$Effect = (33.48+33.48+35)1.2$$
$$A_E = 122mm^2$$

(B) <u>TENSION CAPACITY</u> – (AS4600 Section 3.2)

$$N^* \le \phi A_g f_y \text{ or } \phi(.85k_t A_n F_u)$$
$$\phi = .9 \quad A_g = 163 \quad A_n = 163-(18x1.2) = 141mm^2$$
$$f_y = 500 \quad f_u = 550 \quad k_t = 1.0$$
$$N_t^* = .9x.85x122x550$$
$$= 51.3$$
$$or$$
$$N_t^* = .9x.163x500 = 73.35$$

$$\boxed{N_t^* \le 51.3kN}$$

(C) <u>COMPRESSION CAPACITY</u> - (AS/NZS4600 Section 3.4)

$$N^* \le \phi_c N_s \quad \text{or} \quad \phi_c N_c$$

$$\text{(i)} \quad \phi_c = .85 \quad N_s = A_e f_y = 141x500$$
$$\Rightarrow \phi_c N_s = .85x141x500$$
$$= 59.92kN$$

(ii) Determine Section Properties (Full)

$$I_{xx} \approx 2(36x1.2x31.5^2) + \frac{62^3 x1.2}{12}$$
$$= 109.56x10^3 mm^4$$
$$A = 163 \Rightarrow r_{xx} = \sqrt{\frac{10956}{163}} = 25.9$$

(iii) Determine λ_c (Section 3.4.2)

$$f_{oc} = \frac{\pi^2 E}{\left(\frac{2700}{25.9}\right)^2}$$
$$= 182MPa$$

$$\therefore \lambda_c = \sqrt{\frac{500}{182}} = 1.657 - - - - - - - - - - Eqn\,3.4.1(5)$$

$$\therefore f_n = \left(.877/\lambda^2\right) f_y - - - - - - - Eqn\,3.4.1(3)$$

$$= .319x500 = 159$$

$$\Rightarrow \phi_c N_c = .85x141x159$$

$$= 19.06kN$$

$$\therefore \boxed{N_c^x \le 19.06kN}$$ --- Per Stud

<u>CHECK BOLT TEAR OUT</u> (Bearing) - (AS/NZS4660 Clause 5.3.4.2)

$$V_b^* = \phi V_b$$

$$\phi \propto Cd_f t f_u$$

$$\phi = .6 \qquad\qquad C = 4 - .1\left(\frac{16}{1.2}\right) = 2.667$$

$$\propto \ (Washers\ Built\ in)\ Table\,5.3.4.2(A) = 1.0$$

$$t = 1.2 \qquad f_u = 550$$

$$\therefore V_b^* = .6x2.667x16x1.2x500$$

$$\boxed{V_b^x \le 16.99kN}$$

SUMMARY PER STUD
$N_t^* \le 51.3kN$
$N_c^* \le 59.9kN$
$V_b^* \le 16.99$

Bottom Bracket

$$40x.8\ \ Material$$
$$\therefore N_t^* \le \phi A_g f_y$$
$$\phi = .9 \quad A_g = 25x8 = 200 \quad f_y = 300$$
$$= 54kN$$

$$\boxed{N_t^* \le 54kN}$$

Bolting to Bearer

$$2M10\ Bolts$$
$$N_{tf}^* \le \phi N_{tf} \qquad (AS4100\ 9.3.2.2)$$
$$\phi = .8 \quad f_{uf} = 400 \quad A_s = 60$$
$$N_{tf}^* \le .8x400x60$$
$$\le 19.2kN / Bolt \Rightarrow 38.4kN\ \ for\ 2\ Bolts$$

$$V_f^* \le \phi V_f \left(\phi .62 f_{uf} kr\left(A_c n + nxA_o\right)\right)$$
$$\phi = .8 \quad f_{uf} = 400 \quad n = 1 \quad A_c = 78$$
$$n_x = 0$$
$$\Rightarrow \phi V_F = .8x.62x400x78$$
$$= 15.5kN / Bolt$$
$$\Rightarrow V_F^* \le 31kN$$

$$\boxed{Bottom\ Bracket\ \phi N_t^* = 38.4kN, \phi V_f = 31kN}$$

D. Bearer/ Stump Capacity- JCU – CTS Load path Test Report TS837

The static tensile strength testing was performed generally in accordance with *Clause B3: Prototype Testing* in Appendix B of *AS/NZS 1170.0: Structural Design Actions Part 0: General Principles.* The tensile load was increased slowly until failure was induced in the test specimen.

A maximum tensile load of 55.9 kN was applied to the test specimen.

Table 2: Static Tensile Strength Testing Result

Trial No.	Date Tested	Maximum Load Applied (kN)	Observations
1	4 Aug 2011	55.9	**Failure.** Tear in wall panel stud section material. Loss of load carrying capacity. Bending of bearer top flange.

NOTE: Once bearer flange bends material goes into tension and bolt tear out occurs. Using AS/NZS4600 (5.3.4.2).

These Test results indicate that this bearer fixing is acceptable.

BEARER / STUMP CONNECTION

As tested in JCU-CTS Load path Test Report TS837 the Tensile test was a panel assembly which is roof bolt – to panel – to floor bearer to stump using the Force 10 proprietary brackets.

From this testing, the nominal tensile load (uplift) of the assembly is 45.3 kN.

For a capacity reduction factor of 0.65 ⇒ tensile section capacity is 29.5 kN.

This value is characteristic of the system and requires no further review.

∴ **Section Uplift Capacity - 29.5kN**

CHEMICAL ANCHOR CAPACITY

Consider 3 cases:

a. M12 Chemical anchors 55 mm edge distance N20 Concrete
b. M12 Chemical anchors 125 mm edge distance N20 Concrete
c. M16 Chemical anchors 125 mm edge distance N25 Concrete

a. M12 /N20 /55 – From Ramset Chem Std Limit State Capacities per Anchor (Tension and Shear)

$$\frac{N*}{\phi N_{ur}} + \frac{V*}{\phi V_{ur}} = \leq 1.2 - FromBrochure$$

$$\phi Nur = \phi Nu \cdot Xtc \cdot Xte \cdot Xta$$
$$\phi Nur = 22.3kN$$
$$Xtc = .89$$
$$Xte = 1.0$$
$$Xta = N / A$$
$$\Rightarrow \phi Nur = .89x22.3 = 19.85kN$$

$$\phi Vur = \phi Vu \cdot Xsc \cdot Xse \cdot Xsa$$
$$\phi Vur = 17.2kN$$
$$Xsc = .82$$
$$Xse = .36$$
$$sa = N / A$$
$$\therefore \phi Vur = .17.2x.82x.36 = 5.08kN$$

$$\boxed{\therefore \frac{N*}{19.85} + \frac{V*}{5.08} = \leq 1.2 - 55\textbf{mm Edge distance}}$$

b. M12 /N20 /125 – From Ramset Chem Std Limit State Capacities per Anchor (Tension and Shear)

$$\phi Nur = \phi Nu \cdot Xtc \cdot Xte \cdot Xta$$
$$\phi Nur = 22.3kN$$
$$Xtc = .89$$
$$Xte = 1.0$$
$$Xta = N / A$$
$$\Rightarrow \phi Nur = .89x22.3 = 19.85kN$$

$$\phi Vur = \phi Vu \cdot Xsc \cdot Xse \cdot Xsa$$
$$\phi Vur = 17.2kN$$
$$Xsc = .82$$
$$Xse = 1.0$$
$$sa = N / A$$
$$\therefore \phi Vur = .17.2x.82x1.0 = 14.18kN$$

$$\boxed{\therefore \frac{N*}{19.85} + \frac{V*}{14.1} = \leq 1.2 - 125\textbf{mm Edge distance}}$$

c. M16 /N25 /125 – From Ramset Chem Std Limit State Capacities per Anchor (Tension and Shear)

$$\phi Nur = \phi Nu \cdot Xtc \cdot Xte \cdot Xta$$
$$\phi Nur = 32.6kN$$
$$Xtc = .95$$
$$Xte = 1.0$$
$$Xta = N / A$$
$$\Rightarrow \phi Nur = .95x32.6 = 30.97kN$$

$$\phi Vu = Vu \cdot Xsc \cdot Xse \cdot Xsa$$
$$\phi Vur = 52.3$$
$$Xsc = .91$$
$$Xse = .58$$
$$sa = N / A$$
$$\therefore \phi Vur = 52.3x.91x.58 = 27.6kN$$

$$\boxed{\therefore \frac{N*}{30.97} + \frac{V*}{27.6} = \leq 1.2 - 125\textbf{mm Edge distance M16}}$$

Design Loads N*/V*

a. Design Loads are as per cyclonic loads as per AS4055 and can be combined using net pressure coefficients as
Wall Cpn = 1.35 and Roof Cpn= 1.6

The method for determining maximum uplift is to consider wall loads and put into interaction formula to determine maximum uplift load

$$\frac{N*}{\phi N_{ur}} + \frac{V*}{\phi V_{ur}} =\leq 1.2$$

$$\therefore N* \leq (1.2- = (1.2 - \frac{V*}{\phi V_{ur}})\phi N_{ur}$$

b. V* =can be calculated for C1, C2, C3, C4 conditions and based on **3.0m Panel**

$$\therefore V* = Cpn\frac{PanelHt}{2}$$

$$= 1.35qzx1.5$$

$$or = 2.025qz$$

$\therefore$ THIS CAN BE TABULATED

NCC WIND CATEGORY	qz kPa	V* = 2.025qz
C1	1.5	3.038
C2	2.233	4.522
C3	3.286	6.654
C4	4.438	8.99

NZBC WIND CATEGORY	qz kPa	V* = 2.025qz
TC1	2.233	4.522

$$N* \leq (1.2- = (1.2 - \frac{V*}{\phi V_{ur}})\phi N_{ur}$$

a. For M12/N20/55 $\phi V_{ur} = 5.08$ $\phi N_{ur} = 19.85$

$$\therefore N* \leq (1.2- = (1.2 - \frac{V*}{5.08})19.5$$

b. For M12/N20/125 $\phi V_{ur} = 14.1$ $\phi N_{ur} = 19.85$

$$\therefore N* \leq (1.2- = (1.2 - \frac{V*}{14.1})19.5$$

c. For M12/N20/125 $\phi V_{ur} = 27.6$ $\phi N_{ur} = 30.97$

$$\therefore N* \leq (1.2- = (1.2 - \frac{V*}{27.6})30.97$$

CASE	NCC WIND CATEGORY	V* kN	MAX N* kN
a	C1 C2 C3 C4	3.038 4.522 6.554 8.99	11.95 6.15 N/A N/A
b	C1 C2 C3 C4	3.038 4.522 6.654 8.99	19.54 17.45 14.45 11.16
c	C1 C2 C3 C4	3.038 4.522 6.654 8.99	33.75* 32.1* 29.7* 26.92
***Capacity to be limited to 29.5 kN as per load tests JCU-CTS**			

CASE	NZBC WIND CATEGORY	V* kN	MAX N* kN
a	TC!	4.522	11.95
b	TC!	4.522	17.45
c	TC!	4.522	32.1
***Capacity to be limited to 29.5 kN as per load tests JCU-CTS**			