Force 10 International
Structural Design Calculations
Australia and New Zealand

Prepared By:
W J Dalton
W J Dalton and Associates
U 5, 91 Landsborough Ave
SCARBOROUGH, QLD, 4020

Note: This document is subject to revision and updates are available on request from Force 10 International Pty Ltd.

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<tr>
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<td>24 August 2011</td>
<td>Peter Lehrke</td>
<td>WJ Dalton</td>
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<td>November 2012</td>
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Ref: C:\DOCS\Force 10\Manualls\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 thorough 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:

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

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:

<table>
<thead>
<tr>
<th>DESIGN CAPACITY</th>
<th>STANDARD Up to C3</th>
<th>C4</th>
<th>IPL4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Racking Resistance (kN/Panel)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) 2435 high</td>
<td>5.85</td>
<td>6.5</td>
<td>6.5</td>
</tr>
<tr>
<td>(b) 2700 high</td>
<td>5.27</td>
<td>5.7</td>
<td>5.7</td>
</tr>
<tr>
<td>(c) 3000 high</td>
<td>4.75</td>
<td>5.2</td>
<td>5.2</td>
</tr>
</tbody>
</table>
Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:

Cyclones, Earthquakes, Fire, Termites, Tornadoes

### DESIGN CAPACITY

<table>
<thead>
<tr>
<th>Bending kPa - Static</th>
<th>STANDARD</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Up to C3</td>
<td>C4</td>
</tr>
<tr>
<td>(a) 2435 high</td>
<td>6.79</td>
<td>8.74</td>
</tr>
<tr>
<td>(b) 2700 high</td>
<td>5.52</td>
<td>7.10</td>
</tr>
<tr>
<td>(c) 3000 high</td>
<td>4.47</td>
<td>5.76</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Panel Type</th>
<th>Wind Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Class 1 &amp; 10</td>
</tr>
<tr>
<td>Standard Panel</td>
<td></td>
</tr>
<tr>
<td>(a) 2435</td>
<td>N6 / C4</td>
</tr>
<tr>
<td>(b) 2700</td>
<td>N6 / C3*</td>
</tr>
<tr>
<td>(c) 3000</td>
<td>N6 / C3</td>
</tr>
<tr>
<td>C4 Design</td>
<td></td>
</tr>
<tr>
<td>(a) 2435</td>
<td>N6 / C4</td>
</tr>
<tr>
<td>(b) 2700</td>
<td>N6 / C4</td>
</tr>
<tr>
<td>(c) 3000</td>
<td>N6 / C3</td>
</tr>
<tr>
<td>IPL4 Design</td>
<td></td>
</tr>
<tr>
<td>(a) 2435</td>
<td>N6 / C4</td>
</tr>
<tr>
<td>(b) 2700</td>
<td>N6 / C4</td>
</tr>
<tr>
<td>(c) 3000</td>
<td>N6 / C3</td>
</tr>
</tbody>
</table>

* Updated November 2012

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

<table>
<thead>
<tr>
<th>Panel Type</th>
<th>Wind Classification - Region</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A6 and A7</td>
</tr>
<tr>
<td>Standard Panel</td>
<td></td>
</tr>
<tr>
<td>(a) 2435</td>
<td>TC1</td>
</tr>
<tr>
<td>(b) 2700</td>
<td>TC1</td>
</tr>
<tr>
<td>(c) 3000</td>
<td>TC1</td>
</tr>
<tr>
<td>C4 Design</td>
<td></td>
</tr>
<tr>
<td>(a) 2435</td>
<td>TC1</td>
</tr>
<tr>
<td>(b) 2700</td>
<td>TC1</td>
</tr>
<tr>
<td>(c) 3000</td>
<td>TC1</td>
</tr>
<tr>
<td>IPL4 Design</td>
<td></td>
</tr>
<tr>
<td>(a) 2435</td>
<td>TC1</td>
</tr>
<tr>
<td>(b) 2700</td>
<td>TC1</td>
</tr>
<tr>
<td>(c) 3000</td>
<td>TC1</td>
</tr>
</tbody>
</table>

**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.
Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:
Cyclones, Earthquakes, Fire, Termites, Tornadoes

FLOOR DESIGN

BEARER

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MEMBER CAPACITIES ---------------------------------------------------------------Page F2
SPANS -----------------------------------------------------------------------------Page F5

FLOOR JOIST

SECTION PROPERTIES---------------------------------------------------------------Page F10
MEMBER CAPACITIES ---------------------------------------------------------------Page F10
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Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:
Cyclones, Earthquakes, Fire, Termites, Tornadoes

FLOOR BEARER – SECTION PROPERTIES

\[ I_y = 2 \left( \frac{108^3 \times 2}{12} \right) = 42 \times 10^6 \text{mm}^3 \quad r_y = 19.3, r_s = 65.5 \]

\[ I_{x\ full\ section} = 2 \left( 110 \times 2 \left( \frac{17^3}{2} \right) \right) + 2 \left( \frac{169^3 \times 2}{12} \right) = 4.825 \times 10^6 \text{mm}^4 \]

\[ Z_s = 55.78 \times 10^3 \text{mm}^3 \quad A = 1124 \quad r_v = 68.3 \]

\[ I_w = \frac{d^2 I_y}{4} = 3.07 \times 10^9 \quad J = W_r \frac{t^3}{3} = 1500 \]

Maximum Shear in Web - (Section 3.3.4)

\[
\frac{d_1}{t} = \frac{171}{2} = 85.5
\]

\[
\sqrt{\frac{E_k w}{f_y}} = \sqrt{\frac{2 \times 10^5 \times 5.34}{550}} = 44.07
\]

\& \ 1.415 \sqrt{\frac{E_k w}{f_y}} = 62.35

As \ \frac{d_1}{t} > 62.35 \quad V_r = \frac{0.905 E_k w t^3}{d}

\[
= \frac{0.905 \times 2 \times 10^5 \times 5.34 \times 2^3}{171} = 45.22 \text{kN Per Web}
\]

\[
\therefore \phi V_r = (2 \times 45.22) = 81.4 \text{kN}
\]

\[ \phi V_r = 81.4 \text{kN} \]
Member Moment Capacity - (Section 3.3.3)

\[ \lambda_y = \sqrt{\frac{M_y}{M_o}} \]

\[ M_y = 450 \times 55.78 \times 10^3 \]

\[ = 25.1 kNm \]

\[ M_o = C_b A r_o \sqrt{f_{oy} f_{sc}} \]

\[ C_b = 1.0 \quad A = 1124 \quad r_o = 68.3 \]

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

\[ = \frac{\pi^2 \times 2 \times 10^5}{\left( \frac{500}{19} \right)^2} = 2940 \]

\[ f_{sc} = \frac{GJ}{Ar_o^2} \left[ 1 + \frac{\pi^2 EI_w}{GJl^2} \right] \]

\[ = \frac{80000 \times 1500}{1124 \times 68.3} \left[ 1 + \frac{\pi^2 \times 2 \times 10^5 \times 3.07 \times 10^9}{80000 \times 1500 \times 500^2} \right] \]

\[ = 4623 \]

\[ \therefore M_o = 1.0 \times 1124 \times 68.3 \sqrt{4623 \times 2940} \]

\[ = 284 kNm \]

\[ \therefore \lambda_y = \sqrt{\frac{30.67}{284}} = .33 \]

\[ \Rightarrow M_e = M_y \]

\[ \therefore \phi_e M_e = .9 \times 25.1 \]

\[ = 27.6 kNm \]

\[ \phi_e M = 22.58 kNm \]

\[ \therefore \text{Clause 3.3.5 – Combined Bending / Shear} \]

\[ \left( \frac{M^*}{22.58} \right)^2 + \left( \frac{V^*}{81.4} \right)^2 \leq 1.0 \]
Web Crippling – Section 3.3.6

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

Using Table 3.3.6.2(B)

\( C = 4, \ C_r = .14, \ C_1 = .35, \ C_w = .02, \ \phi_w = .85 \)
\( r_i = 4.0, \ \ell = 2.0, \ \ell_b = 37, \ d_i = 171, \ \sigma = 90^\circ \)

\[ \therefore R_b = 4 \times 2^2 \times 550 \left( 1 - .14 \sqrt{\frac{4}{2}} \right) \left( 1 + .35 \sqrt{\frac{37}{2}} \right) \left( 1 - .02 \sqrt{\frac{171}{2}} \right) \]

\[ = 8800(8)(2.505)(815) \]
\[ = 14.37kN/\text{Web} \]
\[ R_b = 28.745 \]
\[ \therefore \phi R_b = .75 \times 28.745 \]
\[ = 21.6 \]

\[ \phi R_b = 21.6kN \]

\[ \therefore \text{Clause 3.3.7 – Combined Bending / Bearing} \]

\[ 0.82 \left( \frac{R^*}{21.6} \right) + \left( \frac{M^*}{22.58} \right) \leq 1.32 \] (3.3.7(2))

OR

\[ \frac{R^*}{34.77} + \frac{M^*}{29.81} \leq 1.0 \]

Single Bearer Span – 3000 – 1.5 kPa LIVE LOAD

(1) Design Loads
- Strength \( 1.2 \times 7 + 1.5 \times 2 = 3.84 \text{ kPa} \)
- Serviceability \( 0.7 \times 7 + 1.5 = 1.75 \text{ kPa} \)

(2) Design Load Width – 2000 either side =

\[ w_{ult} = 15.36 \text{ kN/m} \]
\[ w_{ser} = 8.4 \text{ kN/m} \]
Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:
Cyclones, Earthquakes, Fire, Termites, Tornadoes

Combined Bending / Shear Check

\[
\frac{(17.28)^2}{(22.58)} + \left(\frac{23}{81.4}\right)^2 = 0.66 < 1.0
\]

Combined Bending / Bearing

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

To Achieve 1.0 Try

\[
M^* = 29.81 - \frac{V^*}{34.77}
\]

\[
M^* = 29.81 - 0.86V^*
\]

ie.

\[
\frac{w\ell^2}{8} = 29.81 - 0.86 \frac{w\ell}{2}
\]

\[
\ell^2 \cdot 15.36 = 29.81 - 0.86 \frac{15.36}{2}
\]

\[
\ell = 1.92\ell^2 + 6.58\ell - 29.81 = 0
\]

\[
\ell = \frac{-6.58 \pm \sqrt{6.58^2 + 4 \times 1.92 \times 29.81}}{2 \times 1.92}
\]

\[
= \frac{-6.58 + 16.5}{3.84}
\]

\[
= 2.58 \text{m}
\]

\[
\therefore \text{Max Span for 2m Load Both Sides} = 2.58 \text{m}
\]
Deflection \( \Delta = \frac{5w\ell^4}{384EI} \)

\[ \Delta = \frac{5 \times 7.0 \times 2740^4}{384 \times 2 \times 10^3 \times 4.825 \times 10^6} \]

\[ = 5.3 \text{mm} \Rightarrow \frac{\text{Span}}{510} \]

\[ \ell = 3 \sqrt[3]{\frac{384EI}{600 \times 5 \times 7}} \]

\[ \ell = 2603 \]

\[ \therefore \text{For Joists either side 4000, Max Span} = 2603 \]

(b) Note: For Joists 3000 either side

\[ \Rightarrow W_{\text{sen}} = 5.25 \text{kN/m and for} \frac{\text{Span}}{600} \]

\[ \Rightarrow \ell = 3 \sqrt[3]{\frac{384EI}{600 \times 5 \times 5.25}} \]

\[ = 2860 \text{ - Still less than 3 Modules} \]

(c) Joists at 2000 either side

\[ W = 3.5 \text{kN/m} \]

\[ \ell = 3 \sqrt[3]{\frac{384EI}{600 \times 5 \times 3.5}} \]

\[ = 3280 \Rightarrow 3 \text{ Modules} \]

(d) Joist 1500 either side

\[ W = 1.75 \times 1.5 = 2.625 \]

\[ \ell = 3 \sqrt[3]{\frac{384EI}{600 \times 5 \times 2.625}} \]

\[ = 3610 \]

**Summary**

<table>
<thead>
<tr>
<th>Joist Spacing</th>
<th>4000</th>
<th>3000</th>
<th>2000</th>
<th>1500</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span</td>
<td>2600</td>
<td>2860</td>
<td>3280</td>
<td>3610</td>
</tr>
</tbody>
</table>

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

**Performance Values**

<table>
<thead>
<tr>
<th>Joist Spacing</th>
<th>4000</th>
<th>3000</th>
<th>2000</th>
<th>1500</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span</td>
<td>2500</td>
<td>3000</td>
<td>3500</td>
<td>4000</td>
</tr>
</tbody>
</table>

**Allowable spans 1.5 kPa LIVE LOAD**
Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:
Cyclones, Earthquakes, Fire, Termites, Tornadoes

**Single Bearer Span – 3000 – 2.0 kPa LIVE LOAD**

(1) Design Loads  
- Strength  
  \[ 1.2 \times 7 + 2.0 \times 2 = 4.84 \, kPa \]
- Serviceability  
  \[ 0.7 + 0.7 \times 2.0 = 2.1 \, kPa \]

(2) Design Load Width – 2000 either side =  
\[ w_{ult} = 19.36 \, kN/m \]
\[ w_{ser} = 8.4 \, kN/m \]

(3) 
\[ M^* = 19.36 \times \frac{9}{8} \]
\[ = 21.78 kNm \]
\[ V^* = 19.36 \times \frac{3}{2} \]
\[ = 29.0 kN \]

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

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

\[ M^* = 29.81 - 0.86V^* \]

ie.  
\[ \frac{w\ell^2}{8} = 29.81 - 0.86 \times \frac{w\ell}{2} \]
\[ \ell^2 \times \frac{19.36}{8} = 29.81 - 0.86x \frac{19.36}{2} \ell \]
\[ 2.42\ell^2 = 29.81 - 8.32\ell \]

OR  
\[ 2.42\ell^2 + 8.32\ell - 29.81 = 0 \]

\[ \ell = \frac{-8.32 \pm \sqrt{8.32^2 + 4 \times 2.42 \times 29.81}}{2 \times 2.42} \]
\[ = \frac{-8.32 + 18.915}{4.84} \]
\[ = 2.18m \]

\[ \therefore \text{Max Span for 2m Load Both Sides} = 2.18m \]
Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:
Cyclones, Earthquakes, Fire, Termites, Tornadoes

\[ \Delta = \frac{5WL^4}{384EI} \]

a. Deflection \( \Delta = \frac{5 \times 8.4 \times 2740^4}{384 \times 2 \times 10^7 \times 4.825 \times 10^6} \)

\[ = \frac{6.4mm}{430} \Rightarrow \text{Span} \]

\[ = 2700 \text{ - Still less than 3 Mod} \]

b. For Joists either side 4000, Max Span = 2380
Note: For Joists 3000 either side

\[ \Rightarrow W_{\text{min}} = 6.3kN/m \text{ and for } \frac{\text{Span}}{600} \]

\[ \Rightarrow \ell = 3 \sqrt[3]{\frac{384EI}{600 \times 5 \times 6.3}} \]

\[ = 3092 \Rightarrow 3 \text{ Modules} \]

c. Joists at 2000 either side
\[ W = 4.2kN/m \]

\[ \Rightarrow \ell = 3 \sqrt[3]{\frac{384EI}{600 \times 5 \times 4.2}} \]

\[ = 2000 \Rightarrow 3 \text{ Modules} \]

d. Joist 1500 either side
\[ W = 2.1 \times 1.5 = 3.15kN/m \]

\[ \Rightarrow \ell = 3 \sqrt[3]{\frac{384EI}{600 \times 5 \times 3.15}} \]

\[ = 3500 \]

Summary

<table>
<thead>
<tr>
<th>Joist Spacing</th>
<th>4000</th>
<th>3000</th>
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<th>1500</th>
</tr>
</thead>
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<tr>
<td>Beam Span</td>
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<td>2700</td>
<td>3000</td>
<td>3500</td>
</tr>
</tbody>
</table>

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

Performance Values

<table>
<thead>
<tr>
<th>Joist Spacing</th>
<th>4000</th>
<th>3000</th>
<th>2000</th>
<th>1500</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span</td>
<td>2500</td>
<td>3000</td>
<td>3000</td>
<td>4000</td>
</tr>
</tbody>
</table>

Allowable spans 2.0 kPa LIVE LOAD
**Single Bearer Span – 3000 – 3.0 kPa LIVE LOAD**

(1) **Design Loads**
- **Strength**: \[1.2 \times 0.7 + 2.0 \times 3.0 = 5.34 \text{ kPa}\]
- **Serviceability**: \[0.7 \times 0.7 \times 3.0 = 2.8 \text{ kPa}\]

(3) **Design Load Width – 2000 either side**

\[w_{ult} = 21.36 \text{ kN/m}\]
\[w_{ser} = 11.2 \text{ kN/m}\]

(3) **Try 2500 Span**

\[M^* = 21.36 \times \frac{2.5^2}{8} = 16.68 \text{ kNm}\]
\[V^* = 21.36 \times \frac{2.5}{2} = 26.7 \text{ kN}\]

**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**

\[
\frac{M^*}{29.81} + \frac{V^*}{34.77} \leq 1.0
\]

\[
\frac{26.7}{34.77} + \frac{16.88}{29.81} = 1.33 > 1.0
\]

To Achieve 1.0 Try \[
M^* = \frac{29.81 - 0.86V^*}{29.81}
\]

ie. \[
\frac{w\ell^2}{8} = 29.81 - 0.86 \times \frac{w\ell}{2}
\]

\[
\ell^2 \times \frac{21.36}{8} = 29.81 - 0.86 \times \frac{21.36}{2} \ell
\]

\[
= 2.67\ell^2 = 29.81 - 9.18\ell
\]

OR \[
2.67\ell^2 + 9.18\ell - 29.81 = 0
\]

\[
\ell = \frac{-9.18 \pm \sqrt{9.18^2 + 4 \times 2.67 \times 29.81}}{2 \times 2.267}
\]

\[
= -9.18 + 20.07
\]

\[
= 5.34
\]

\[
= 2.04 \text{ m}
\]

\[\therefore\] Max Span for 2m Load Both Sides = 2.04m
Deflection \( \Delta = \frac{5w\ell^4}{384EI} \)

\[ \Delta = \frac{5 \times 11.2 \times 2000^4}{384 \times 2 \times 10^3 \times 4.825 \times 10^6} = 2.4\text{mm} \Rightarrow \frac{\text{Span}}{800} \]

(b) For Joists 3000 either side

\[ \Rightarrow W_{\text{sen}} = 8.4\text{kN/m} \text{ and for } \frac{\text{Span}}{600} \]

\[ \Rightarrow \ell = 3\sqrt[4]{\frac{384EI}{600 \times 5 \times 8.4}} = 2450 - \text{Still less than 3 Mod} \]

(c) Joists at 2000 either side

\[ W = 5.6\text{kN/m} \]

\[ \Rightarrow \ell = 3\sqrt[4]{\frac{384EI}{600 \times 5 \times 5.6}} = 2800 \Rightarrow 3 \text{ Modules} \]

(d) Joist 1500 either side

\[ W = 4.2\text{kN/m} \]

\[ \Rightarrow \ell = 3\sqrt[4]{\frac{384EI}{600 \times 5 \times 4.2}} = 3090 \]

**Summary**

<table>
<thead>
<tr>
<th>Joist Spacing</th>
<th>4000</th>
<th>3000</th>
<th>2000</th>
<th>1500</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span</td>
<td>2000</td>
<td>2450</td>
<td>2800</td>
<td>3090</td>
</tr>
</tbody>
</table>

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

**Performance Values**

<table>
<thead>
<tr>
<th>Joist Spacing</th>
<th>4000</th>
<th>3000</th>
<th>2000</th>
<th>1500</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Span</td>
<td>2000</td>
<td>2500</td>
<td>3000</td>
<td>3000</td>
</tr>
</tbody>
</table>

Allowable spans 3.0 kPa LIVE LOAD
FLOOR JOISTS – G500

SECTION PROPERTIES – FULL SECTION

\[ A = 360.2 \quad I_x = 1.63 \times 10^6 \quad Z_x = 34.32 \times 10^3 \quad r_x = 67.3 \]
\[ J = 187.6 \quad I_y = 0.0764 \times 10^6 \quad Z_y = 3.056 \times 10^3 \quad r_y = 14.6 \]

Maximum Shear – In Web – (Section 3.3.4)

\[
\frac{d_y}{t} = \frac{95}{1.2} = 79.17, \quad \sqrt{\frac{E_k}{f_y}} = \sqrt{\frac{2 \times 10^3 \times 5.34}{500}} = 46.22
\]

\[
1.415 \sqrt{\frac{E_k}{f_y}} = 65.40
\]

\[
\therefore \text{Where} \quad \frac{d_y}{t} > 1.415 \quad \sqrt{\frac{E_k}{f_y}}, \quad V_v = \frac{905E_k t^3}{d_y}
\]

\[
\therefore V_v = 17.58 \text{kN}
\]

\[
\therefore \phi V_v = 9 \times 17.58
\]

\[
= 15.823 \text{kN}
\]

\[
\phi V_v = 15.82 \text{kN}
\]

Member Moment Capacity – (Section 3.3.3)

\[
\lambda_b = \sqrt{\frac{M_y}{M_o}}
\]

\[ M_y = 500 \text{MPa} \quad M_o = \frac{\pi^2 E C_b d I_{wc}}{2 \ell^2} \]

\[ C_b = 1.0 \quad I_{wc} = \frac{I_x}{2} \quad d = 95 \quad \ell = 3000 \]

\[
= M_o = \frac{\pi^2 \times 2 \times 10^5 \times 1.0 \times 95 \times 815 \times 10^6}{2 \times 3000^2}
\]

\[
= 8.5 \text{kNm}
\]

\[ M_y = 500 \times 34.32 \times 10^3 = 17.16 \]
\[ \lambda_b = \sqrt{\frac{17.16}{8.5}} = 1.42 > 1.336 \]  
(Clause 3.3.3.2(4))

\[ M_c = M_y \left( \frac{1}{\lambda_b^2} \right) \]

\[ 17.16 \times 0.495 = 8.5 \, kNm \]

\[ \phi_M M_c = 0.9 \times 8.5 \]

\[ = 7.65 \]

\[ \phi_M M_o = 7.65 \, kNm \]

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

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

**Spans**

- **Design Loads**

  (1) **Dead Load** = 0.7kPa  
  **Live Load** = 2.0kPa

\[ \therefore w = 1.2 \times 0.7 + 1.5 \times 2 \]

\[ = 3.84kPa \]

\[ = 3.84 \times 5kN/m \]

\[ = 19.2kN/m \]

\[ M^* = 1.92 \times 3^2 \times \frac{3}{8} = 2.16kNm \]

\[ V^* = 1.92 \times 3^2 \times \frac{3}{2} = 2.88kN \]

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

\[ = 0.11 \]
(2) Serviceability

\[ w = 0.7 + 0.7 \times 2 \quad (g + \Psi, q) \]
\[ = 2.1 \text{kPa} \]
\[ \Rightarrow 1.05 \text{kN/m} \]
\[ \Delta = \frac{5wl^4}{384EI} \]
\[ = \frac{5 \times 1.05 \times 3000^4}{384 \times 2 \times 10^3 \times 1.63 \times 10^6} \]
\[ = 3.397 \text{mm} \]

ie. \( \frac{\text{Span}}{880} < \frac{\text{Span}}{600} \) OK

\[ :.; \ 1.2 \text{mm JOIST OK TO 3000 SPAN} \]

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

\[ l = \sqrt{\frac{384EI}{600 \times 5 \times 1.05}} \]
\[ = 3900 \text{ ie. Close enough to 4.0 mod} \]
DYNAMIC PERFORMANCE – LIGHT FLOOR SYSTEMS

AS3623 – APPENDIX D

Response to /kN Static Load - Section D2

Bearer Span - 3500
Load Width - 2500

CALCULATE \( P_d \)

\[
k_b = \frac{E_b I_b}{\ell^3}
\]

\[
= \frac{2 \times 10^5 \times 4.825 \times 10^6}{3500^3}
\]

\[
= 22.51
\]

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

\[
= \frac{3000 \times 19^3 \times 3500}{12 \times 500^3}
\]

\[
= 48.0
\]

\[
\therefore \frac{k_b}{k_c} = 2.13
\]

FIGURE D1 => \( P_d = .8 \)

\[
\therefore P_d = .8 \times 1000 = 800N
\]

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

\[
= \frac{800 \times 3500^3}{48 	imes 2 \times 10^5 \times 4.825 \times 10^6}
\]

\[
= .74mm > 1.0 \text{ OK}
\]

Unit Impulse – Section D3

NATURAL FREQUENCY

\[
K_s = \frac{E_s I_b}{S}
\]

\[
= \frac{2 \times 10^5 \times 4.825 \times 10^6}{500}
\]

\[
= 1930 \times 10^6
\]
Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:
Cyclones, Earthquakes, Fire, Termites, Tornadoes

\[ K_y = \frac{E_i t_r^3}{12} \]
\[ = \frac{3000 \times 19^3}{12} \]
\[ = 1.715 \times 10^6 \]
\[ \therefore K_x >> K_y \text{ OK TO USE FORMULAE D3.2} \]

\[ F = \frac{\pi}{2} \sqrt{\frac{K_x}{w f^4}} \]

- **DL** - 30kg/m²
- **LL** - 3kPa, i.e. 30kg/m²

\[ = f = \frac{\pi}{2} \sqrt{\frac{1930 \times 10^6 \times 10^{-3}}{60 \times 3.5^4}} \]
\[ = 23 \]

**Maximum Impact Velocity – Section D4**

\[ K_x >>> K_y \Rightarrow N_{40} = \frac{B}{L} \left( \frac{f^2 - 1}{r} \right)^{\frac{1}{4}} \]

- **B** = 2.5
- **L** = 3.5
- **F** = \( \frac{40}{23} = 1.74 \)

\[ r = \frac{K_y}{K_x} = \frac{1.715}{1930} = 0.0009 \]
\[ N_{40} = \frac{2.5 \left( 1.74 - 1 \right)}{3.5 \left( 0.0009 \right)}^{\frac{1}{4}} \]
\[ = 3.84 \]

\[ V_{\text{max}} = \frac{4 \left( 0.4 + 6.3 \times 3.84 \right)}{60 \times 3.5 \times 2.5 + 200} \]
\[ = 0.149 \text{m/sec (or 15mm/sec)} \]
\[ \log_{10} V_{\text{max}} < 1.2 + 2\sigma \]
\[ 2\sigma = 2 \left( f_1 V \times \frac{9}{100} \right) = 2 \left( \frac{23 \times 9}{100} \right) \]
\[ = 0.414 \]
\[ \therefore 1.2 + 2\sigma = 1.614 \]
\[ \log_{10} 15 = 1.176 < 1.614 \]
\[ \therefore \log_{10} V_m < 1.2 + 2\sigma \]

\[ \therefore \text{FOR 2.5 WIDE FLOOR JOIST EACH SIDE BEARER SPAN} \]
Check Bearing / Bending:

Live Load - 1.5kPa   Dead Load - 0.7kPa

\[ w = 1.5 \times 1.5 + 1.2 \times 7 \]
\[ w = 3.09 \text{kPa} \]

\[ w = 7.725 \text{kN/m} \quad \text{(2.5 Wide)} \]

\[ \text{Span} = 3.5m \Rightarrow M^* = 7.725 \times \frac{3.5^2}{8} \]
\[ M^* = 11.83 \]

\[ R^* = 13.52 \]

\[ \frac{R^*}{34.77} + \frac{M^*}{36.43} \leq 1.0 \]
\[ \frac{13.52}{34.77} + \frac{11.83}{36.43} \]
\[ = 0.39 + 0.32 \]
\[ = 0.71 < 1.0 \]

\[ \therefore \text{Bearer Acceptable} \]
WALL PANEL ANALYSIS

TEST RESULTS---------------------------------------------------------------Page W2
BENDING CAPACITY---------------------------------------------------------------Page W2
RACKING CAPACITY---------------------------------------------------------------Page W4
AXIAL CAPACITY---------------------------------------------------------------Page W5
COMBINED BENDING & COMPRESSION-----------------------------------------------Page W7
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:

<table>
<thead>
<tr>
<th>TEST TITLE</th>
<th>REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind Load Static Strength Test Of Standard Panel</td>
<td>FORT 1101</td>
</tr>
<tr>
<td>Static And Cyclic Wind Load Strength Testing Of IPL4 Wall Panels</td>
<td>Report TS826</td>
</tr>
<tr>
<td>Simulated Windborne Debris Impact Testing Of IPL4 Wall Panels</td>
<td>Report TS829</td>
</tr>
<tr>
<td>Serviceability, Static And Cyclic Racking Testing Of IPL4 Wall Panels</td>
<td>Report TS830</td>
</tr>
<tr>
<td>Static Tensile Strength Testing Of Wall Panel Stud System With Truss And Floor Connections</td>
<td>Report TS837</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Test Reference</th>
<th>Standard</th>
<th>IPL4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending Capacity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Static</td>
<td>6.79 *</td>
<td>7.8</td>
</tr>
<tr>
<td>(b) Cyclic</td>
<td>-</td>
<td>9.7</td>
</tr>
<tr>
<td>Impact test</td>
<td>Passed</td>
<td></td>
</tr>
<tr>
<td>Racking Resistance (3 Panels)</td>
<td>21.4</td>
<td></td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>45.3*</td>
<td>45.3*</td>
</tr>
</tbody>
</table>

*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.

<table>
<thead>
<tr>
<th>Non Cyclonic Ultimate Design Capacity (kPa)</th>
<th>Cyclonic Ultimate Design Capacity (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.79</td>
<td>6.79</td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th>Panel Height (H)</th>
<th>Ratio (2435/H)</th>
<th>Factor (Ratio x Ratio or R²)</th>
<th>Ultimate Bending Capacity (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2700</td>
<td>0.902</td>
<td>0.813</td>
<td>5.43</td>
</tr>
<tr>
<td>3000</td>
<td>0.812</td>
<td>0.659</td>
<td>4.44</td>
</tr>
</tbody>
</table>

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.

<table>
<thead>
<tr>
<th>Class</th>
<th>Wind Classification</th>
<th>Ultimate Wind Speed</th>
<th>Net Pressure Coefficient</th>
<th>Net Pressure P (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vn</td>
<td>Cpn</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-10</td>
<td>C3/N6</td>
<td>74</td>
<td>1.35</td>
<td>4.4</td>
</tr>
<tr>
<td>1-10</td>
<td>C4/N6</td>
<td>86</td>
<td>1.35</td>
<td>6.0</td>
</tr>
<tr>
<td>2-9</td>
<td>C2/N5</td>
<td>61</td>
<td>2.0</td>
<td>4.9</td>
</tr>
<tr>
<td>2-9</td>
<td>C3/N6</td>
<td>74</td>
<td>2.0</td>
<td>6.6</td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th>Panel size</th>
<th>Wind Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>2435</td>
<td>N6/C4</td>
</tr>
<tr>
<td>2700</td>
<td>N6/C3</td>
</tr>
<tr>
<td>3000</td>
<td>N6/C3</td>
</tr>
</tbody>
</table>

C4 /IPL4 Panel:

<table>
<thead>
<tr>
<th>Panel size</th>
<th>Wind Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>2435</td>
<td>N6/C4</td>
</tr>
<tr>
<td>2700</td>
<td>N6/C4</td>
</tr>
<tr>
<td>3000</td>
<td>N6/C3</td>
</tr>
</tbody>
</table>

* 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 $\varnothing V_b = \varnothing \propto C d f_t f_u = (AS/NZS 4600 Cl 5.3.4)$

Where:
- $\varnothing = 0.6$
- $\propto = 1.0$
- $d_t = 16\text{mm}$
- $t = 1.2\text{mm}$
- $f_u = 520\text{Mk}$

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

$C = 4 - .1 \times 13.33 \quad \text{Table 5.3.4 (B)}$

$= 2.667$

$\therefore \varnothing V_b = 0.6 \times 2.667 \times 1.6 \times 1.2 \times 520$

$= 15.98 \text{kN}$

For 2435 x 990 panel, from statics

Racking load $\times 2.435 = \varnothing V_b \times 0.99 \Rightarrow$ Racking load 6.5 kN

For standard panel reduce by 0.9 $\Rightarrow 6.5 \text{kN} \times 0.9 = 5.85 \text{kN}$

**Ultimate Racking Value = 5.85 kN**

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

<table>
<thead>
<tr>
<th>Recommended Ultimate Design Racking Loads</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel Height</td>
<td>Standard (kN /panel)</td>
</tr>
<tr>
<td>---------------</td>
<td>----------------------</td>
</tr>
<tr>
<td>2435</td>
<td>5.85</td>
</tr>
<tr>
<td>2700</td>
<td>5.27</td>
</tr>
<tr>
<td>3000</td>
<td>4.75</td>
</tr>
</tbody>
</table>
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)

<table>
<thead>
<tr>
<th>Series</th>
<th>Test No.</th>
<th>Description</th>
<th>Results</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>CET – 4999/1</td>
<td>1</td>
<td>1 Panel</td>
<td>80.2</td>
<td>60.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1 Panel</td>
<td>84.1</td>
<td>63.1</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1 Panel</td>
<td>83.8</td>
<td>62.8</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>62.0</td>
<td></td>
</tr>
</tbody>
</table>

**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)

<table>
<thead>
<tr>
<th>Series</th>
<th>Test No.</th>
<th>Description</th>
<th>Results</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>1</td>
<td>1 Panel</td>
<td>30.1</td>
<td>22.58</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1 Panel</td>
<td>37.0</td>
<td>27.75</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1 Panel</td>
<td>36.2</td>
<td>27.15</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>25.8</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>1 Panel</td>
<td>28.2</td>
<td>21.15</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1 Panel</td>
<td>37.7</td>
<td>28.28</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1 Panel</td>
<td>36.4</td>
<td>27.30</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>25.58</td>
<td></td>
</tr>
</tbody>
</table>

**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.

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Nominal Member Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive</td>
<td>62.03kN</td>
</tr>
<tr>
<td>Tensile</td>
<td>25.58kN</td>
</tr>
</tbody>
</table>

(Wall Panel Analysis)
E AXIAL LOADS – 3000 HIGH PANELS

Similar testing to the 2700 panels determined the following:

(1) Axial Tests (Compression & Tensile)

<table>
<thead>
<tr>
<th>Series</th>
<th>Test No.</th>
<th>Description</th>
<th>Results</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>1</td>
<td>1 Panel</td>
<td>78.0</td>
<td>58.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1 Panel</td>
<td>74.7</td>
<td>56.03</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1 Panel</td>
<td>75.0</td>
<td>56.25</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td>57.0</td>
<td></td>
</tr>
<tr>
<td>1 - 4</td>
<td>1</td>
<td>1 Panel</td>
<td>39.3</td>
<td>29.47</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1 Panel</td>
<td>44.4</td>
<td>33.3</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1 Panel</td>
<td>44.7</td>
<td>33.5</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td>32.1</td>
<td></td>
</tr>
</tbody>
</table>

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.

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Safe Working Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive</td>
<td>57.0 kN</td>
</tr>
<tr>
<td>Tensile</td>
<td>32.1 kN</td>
</tr>
</tbody>
</table>

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

\[ \frac{N^*}{\phi_c N_c} + \frac{C_{mx} M^*_c}{\phi_b M_{bx} \alpha_{mx}} \leq 1.0 \]  

(3.5.1(1))

From Test Results

\[ N_c = 62kN \quad M_{bx} = 6.79kPa \]
\[ C_{mx} = 0.85 \quad \phi_c = .85 \quad \phi_b = .9 \]
\[ \alpha_{mx} = 1 - \frac{N^*}{N_c} \]
\[ N_c = \frac{\pi^2 EI_b}{(\ell_{eb})^2} \]

\[ E \Rightarrow \text{From Tests} \quad P = 81kN \quad \Delta = 1.2\text{mm} \quad A = 2.6.1000mm^2 \quad I = 2700 \]
\[ E = \frac{P\ell}{A\Delta} = \frac{81 \times 10^3 \times 2700}{12000 \times 1.2} \]
\[ = 15000\text{MPa} \]
\[ I_{eb} = 2\text{Wall sheets separated 65mm} \]
\[ I_{eb} = 2A_{eb}^2 = 2 \times 6000 \times \left( \frac{65}{2} + 3 \right)^2 \]
\[ = 15.12 \times 10^6 \text{mm}^4 \]
\[ \therefore \ell_{eb} = 2700\text{mm} \]
\[ N_c = \frac{\pi^2 EI_b}{(\ell_{eb})^2} = \frac{\pi^2 \times 15000 \times 15.12 \times 10^6}{2700^2} \]
\[ = 307.1kN \]
\[ \therefore \alpha_{mx} = \left( 1 - \frac{N^*}{307} \right) \]

\[ \therefore \text{Eqn 3.5.1(1) becomes} \]
\[ \frac{N^*}{.85 \times 62} + \frac{.85 M^*_c}{.9 \times 6.79 \left( 1 - \frac{N^*}{307} \right)} \leq 1 \]

OR

\[ \frac{N^*}{52.7} + \frac{M^*_c}{7.19 \left( 1 - \frac{N^*}{307} \right)} \leq 1.0 \]
2700 Combined Bending & Compression AS/NZS4600 : P3.5

\[
\frac{N^*}{\phi_c N_e} + \frac{C_{mx}M^*_{bx}}{\phi_b M_{bx} \alpha_{mx}} \leq 1.0 \quad (3.5.1(1))
\]

From Test Results

\[
N_e = 62kN \quad M_{bx} = 5.43kPa
\]

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

\[
\alpha_{mx} = 1 - \frac{N^*}{N_e}
\]

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

\[
E \Rightarrow \text{From Tests} \quad P = 81kN \quad \Delta = 1.2\text{mm} \quad A = 2.6.1000\text{mm}^2 \quad I = 2700
\]

\[
E = \frac{P\ell}{A\Delta} = \frac{81\times10^3 \times 2700}{12000\times1.2}
\]

\[
= 15000\text{MPa}
\]

\[
I_b = 2\text{Wall sheets separated 65mm}
\]

\[
I_b = 2A^2 = 2 \times 6000 \times \left(\frac{65}{2} + 3\right)^2
\]

\[
= 15.12\times10^6\text{mm}^4
\]

\[
\therefore \ell_{eb} = 2700\text{mm}
\]

\[
N_e = \frac{\pi^2 EI_b}{(\ell_{eb})^2} = \frac{\pi^2 \times 15000 \times 15.12\times10^6}{2700^2}
\]

\[
= 307.1kN
\]

\[
\therefore \alpha_{mx} = \left(1 - \frac{N^*}{307}\right)
\]

\[
\therefore \text{Eqn 3.5.1(1) becomes}
\]

\[
\frac{N^*}{.85 \times 62} + \frac{.85M^*}{.9 \times 5.43 \left(1 - \frac{N^*}{307}\right)} \leq 1
\]

OR

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

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

\[ \frac{N^*}{\phi_N N_c} + \frac{C_{mx} M_x^*}{\phi_M M_{bx} \phi_{mx}} \leq 1 \]  
(3.5.1(1))

From Test Results

\[ N_c = 57.0kN \quad M_{bx} = 4.44kPa \]
\[ C_{mx} = 0.85 \quad \phi_c = .85 \quad \phi_b = .9 \]
\[ \phi_{mx} = 1 - \frac{N^*}{N_c} \]
\[ N_c = \frac{\pi^2 E I_b}{(\ell_{ob})^2} \]

\[ E = \text{FROM TESTS} \quad P = 68kN \quad \Delta = .99 \quad A = 2\times6\times1000 \quad \ell = 3000 \]
\[ E = \frac{P\ell}{A\Delta} = \frac{68000\times3000}{12000\times.99} = 17171mm^3 \]
\[ I_b = 2Ay^2 = 2\times6000\times\left(\frac{65}{2} + 3\right)^2 \]
\[ = 15.12\times10^6 \]
\[ \therefore N_e = \frac{\pi^2 x17171 x15.12 x10^6}{3000^2} \]
\[ = 285kN \]
\[ \therefore \phi_{mx} = \left(1 - \frac{N^*}{285}\right) \]

\[ \Rightarrow Eqn \ 3.5.1(1) \Rightarrow \frac{N^*}{.85\times57} + \frac{.85M^*}{.9\times4.44\left(1 - \frac{N^*}{285}\right)} \]

OR

\[ \frac{N^*}{48.5} + \frac{M^*}{4.70\left(1 - \frac{N^*}{285}\right)} \]
Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:
Cyclones, Earthquakes, Fire, Termites, Tornadoes

FORCE10
A better way to build!

ROOF DESIGN

SECTION PROPERTIES ------------------------------------------ Page R2
MEMBER CAPACITIES ----------------------------------------- Page R3
REVIEW OF TRUSS MEMBERS ----------------------------------- Page R7
COMPUTER INPUT LOADS -------------------------------------- Page R8
SECTION PROPERTIES

Warping Constants - Chords

\[ A = 280 \]
\[ I_x = 173.7 \times 10^3 \]

\[ I_w = \frac{b^2 t}{6} \left( 4c^3 - 6ac^2 + 3a^2 c + a^2 b \right) - m^2 I_{xx} \]
\[ m = \frac{a^2 b^2 t}{I_x} \left( \frac{1}{4} + \frac{c}{2b} - \frac{2c^3}{3a^2 b} \right) \]
\[ = \frac{40^3 \times 65 \times 1.2 \left( \frac{1}{4} + \frac{31}{2} - \frac{2}{3} \times 31^3 \right)}{173.7 \times 10^3 \left( \frac{1}{4} + \frac{2}{65} - \frac{3}{40^2} \times 65 \right)} \]
\[ \therefore m = 13.9 \text{mm} \]

\[ I_w = \frac{65^3 \times 1.2}{6} \left( 4 \times 31^3 - 6 \times 40 \times 31^2 + 3 \times 40^2 \times 31 + 40^2 \times 65 \right) - (13.9^2) \times 173.7 \times 10^3 \]
\[ = 845 \times (119164 - 230640 + 148800 + 104000) - 33,560,587 \]
\[ \therefore I_w = 0.86 \times 10^9 \]

\[ J = \frac{bt^3}{3} = \frac{(31 + 31 + 65 + 40) \times 1.2}{3} \]
\[ \therefore J = 75 \]

Warping Constants - Webs

\[ m = \frac{36}{F + 6} \]
\[ F = \frac{a}{b} = 1 \]
\[ = \frac{3 \times 40}{7} \]
\[ = 17.14 \]

\[ I_w = \frac{a^2 b^3 \times 1.2}{12} \left( \frac{2 + 3}{1 + 6} \right) \]
\[ = \frac{40^2 \times 40^3 \times 1.2 \left( \frac{5}{7} \right)}{12} \]
\[ = 7.31 \times 10^6 \]
MEMBER CAPACITIES – ULTIMATE WIND SPEED 60m/s

Load Case 13 DL+WL 90 (One Truss)

Top Chord Worst Case
\( (\ell = 2.129m - \text{Highest Axial Loads}) \)

Based on this orientation
Section Properties are:
\[
\begin{align*}
A &= 280\text{mm}^2 \\
I_w &= 8.4 \times 10^7 \\
I_y &= 173.7 \times 10^3 \\
I_x &= 189.7 \times 10^3 \\
Z_y &= 4.89 \times 10^3 \\
Z_x &= 3.579 \times 10^3 \\
r_y &= 24.9 \\
r_x &= 26.0 \\
r_o &= 38.7 \\
J &= 75
\end{align*}
\]

Nominal Member Moment Capacity (AS4600 Section 3.3.3)

\[
F_c = \frac{M_c}{Z_f}
\]

\( M_c \) depends on \( \lambda_b \)

\[
\lambda_b = \sqrt{\frac{M_y}{M_o}}
\]

\( M_y = F_y Z_y = 500 \times 4.89 \times 10^3 = 2.445 \times 10^6 \text{kNm} \)

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

\( C_b \) can always be 1.0 \( A = 280 \) \( r_o = 38.7 \)

\[
F_{oy} = \frac{\Pi^2 E}{\left( \frac{\ell_y}{r_y} \right)^2} \quad (\ell_y = .75 \ell = 1600)
\]

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

\[
= 478 \text{MPa}
\]
\[ F_{oc} = \frac{\Pi^2 E}{(1600 \frac{26}{26})^2} \]

= 520

\[ F_{oc} = \frac{GJ}{Ar_o^2} \left[1 + \frac{\Pi^2 EI_o}{GJl^3}\right] \]

= \frac{80000 \times 75}{280 \times 1498} \left[1 + \frac{\Pi^2 \times 2 \times 10^4 \times 8.56 \times 10^7}{80000 \times 75 \times 1600^2}\right] \]

= 172 MPa

\[ M_o = 1.0 \times 280 \times 38.7 \sqrt{172 \times 478} \]

= 3.11 kNm

\[ \lambda = \frac{M_o}{M_o} \]

\[ = \frac{2.445}{0.887} \]

\[ = 2.12 kNm \]

\[ f_c = \frac{2.12 \times 10^6}{4.89 \times 10^3} \]

\[ = 434 MPa \]

\[ M_o = Z_F c \]

\[ = 434 \times 4.89 \times 10^3 \]

\[ M_b = 2.12 kNm \]

\[ \phi_b M_b = .9 \times 2.12 \]

\[ = 1.91 kNm \]

BEND PER METRE

\[ \phi_b M_b = 1.91 kNm \]
Nominal Member Compression Capacity 3.4.3

\[
\lambda_c = \sqrt{\frac{r_y}{F_{oc}}}
\]

\( F_y = 550 \)

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

\( \beta = 1 + \left( \frac{r_x}{r_o} \right)^2 \)

\( F_{ox} = 520 \)

\( = 1.45 \quad F_{oc} = 172 \)

\( = 0.3445 \left[ (520 + 172) - \sqrt{(520 + 172)^2 - 4 \times 1.45 \times 520 \times 172} \right] \)

\( = 0.3445 \times (692 - 199) \)

\( = 170 MPa \)

\( \therefore \lambda_2 = \sqrt{\frac{500}{170}} \)

\( = 1.71 \)

\( F_u = \left( \frac{0.877}{1.71^2} \right) 500 \)

\( = 150 MPa \)

\( \therefore N_c = 150 \times 280 \)

\( = 42 \)

\( \phi N_c = 0.85 \times 42 \)

\( = 35.7 \)

\[ \therefore \phi N_c = 35.7 kN \]
Nominal Section Capacity - Tension (3.2.1)

\[ N^* = \phi_t N_T \]
\[ \phi_t = 0.9 \]
\[ N_T = A_n F_y \]
\[ = 280 \times 500 = 140kN \]

or \[ 0.85k_T A_n F_N \]
\[ k_T = 1.0 \quad A_n = 280 \quad F_u = F_y = 550 \]
\[ N_T = 0.85 \times 280 \times 500 \]
\[ = 119kN \]
\[ N^* = 0.9 \times 119 \]
\[ = 107kN \]

\[ \phi_t N_T = 107kN \]

**SUMMARY**

\[ \phi_t M_b = 1.91kNm \]

\[ \phi_t N_c = 35.7kN \]

\[ \phi_t N_T = 107kN \]
TRUSS MEMBERS - Combined

\[
\frac{N^*}{\phi_c N_c} + \frac{C_m M^*}{\phi_b M_o \alpha_{mx}} \leq 1.0
\]

\[
C_m = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right)
\]

\[
M_1 = 0.35, M_2 = 0.75 \text{ - Computer Case 1}
\]

\[
= 0.6 + 0.4 \left( \frac{0.35}{0.75} \right) = 0.79 \text{ (Double Curvature)}
\]

\[
\alpha_{mx} = 1 - \frac{N_1}{N_c}
\]

\[
N_c = \frac{\Pi^2 E I_y}{(\epsilon)^2}
\]

\[
= \frac{\Pi^2 E x 173.7 \times 10^3}{1600^2}
\]

\[
= 134 \text{ kN}
\]

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

\[
\therefore N^* \left( \frac{1}{34} + \frac{0.79 M^*}{2.2 \left( 1 + \frac{N^*}{134} \right)} \right) \leq 1.0
\]

General Formula

\[
\frac{N^*}{34} + \frac{M^*}{2.42 \left( 1 - \frac{N^*}{134} \right)} \leq 1.0
\]
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 $M' = 53kNm$

Neither approach capacity

Webs - Max $N_c = 9.42kN$
(Member 21 – Case 13)
(Length = 1.77m with fixity) ∴ Safe to use 1.6 calculations

∴ $N_c = 34kN$

Reactions

Uplift = 19.1kN
Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:
Cyclones, Earthquakes, Fire, Termites, Tornadoes

**Loads For Computer “C2” or “TC1” Wind Area**

WL 0°
WL 90° (one truss)

**Dead Load**
1.0kPa on Roof \(-0.10kN/m (ult = 0.15)\)
1.0kPa on Bottom Chord \= 0.10kN/m (ult = 0.15)
Wind = 60m/sec => \( q_u = 2.16kPa \)

**Live Load**
\[
\frac{0.12 + \frac{1.8}{11}}{1} = \frac{0.29kPa}{(ULT = 0.44kPa)}
\]

Wind Loads (1000CTS)

**WL0°**
\[-0.3 \times 2.16 = -0.648kN/m\]
\[-0.6 \times 2.16 = -1.296\]
\[+0.7 \times 2.16 = +1.512\]

**WL90°**
\[-0.9 \times 2.16 = -1.944\]
\[+0.7 \times 2.16 = +1.512\]
# COMPONENT FIXINGS

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<tr>
<td>Stud Capacity</td>
<td>C2</td>
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<td>Bottom Bracket</td>
<td>C4</td>
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<td>C5</td>
</tr>
<tr>
<td>Chemical Anchor Capacity</td>
<td>C6</td>
</tr>
</tbody>
</table>

(Component Fixings)
A. Top Bracket

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

(1) **PLATE – Tensile Capacity (AS4100)**

\[ N^* \leq \phi A_y f_y \text{ OR } (0.85 k_t A_n f_y) \phi \]

\[ A_y = 40 \times 8 = 320 \quad f_y = 300 \quad k_t = 1.0 \quad A_n = (40 - 17) \times 8 = 184 \]

\[ f_y = 400 \quad \phi = .9 \quad k_t = 1.0 \]

\[ N^* = 0.9 \times 320 \times 300 \text{ or } (0.85 \times 184 \times 400) \times 9 \]

\[ = 86.4 \text{ or } 56.3kN \]

(2) **M16 STUD - Clause 9.3 (AS4100)**

\[ N_{sf} = \phi N_{TF} \]

\[ \phi = .8 \quad f_{sf} = 400 \quad A_j = 157 \]

\[ \Rightarrow N_{sf} = .8 \times 400 \times 157 \]

\[ = 50.24kN \]

(3) **WELD – Clause 9.7 (AS4100)**

\[ V_w^* = \phi V_w \]

\[ V_w = .6 f_{aw} t_j k_r \]

\[ f_{aw} = 410 \quad t_j = 4.24 \quad k_r = 1.0 \quad \phi = .6 \]

\[ = V_w^* = (.6 \times 410 \times 4.24) \times 6 \]

\[ = .626kN / mm \]

\[ \ell = 32mm \Rightarrow V_w^* = 20.03kN \]

\[ \therefore \text{Top Bracket } \phi V_w^* = 20kN \]
B. Stud Material

G500 material 1.2mm thick

(A) SECTION PROPERTIES

Assume sheeting act as stiffened lip.

\[
\frac{b}{t} = \frac{36 - 6}{1.2} = 25.0
\]

\[
\frac{b}{t} = \frac{64 - 12}{1.2} = 43.3
\]

Effective widths – (AS4600 Clause 2.2.1.2)

\[
\lambda = \sqrt{\frac{f_s}{f_{cr}}}
\]

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

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

\[
f_{cr} = \frac{4 \times 10^5 \times 3.14^2}{12(1-.3^2)} \left( \frac{1.2}{36} \right)^2
\]

= 850

\[
\therefore \lambda = \sqrt{\frac{500}{850}} = .767
\]

\[
\rho = \left( 1 - \frac{25}{850} \right) .767
\]

= .93

\[
\therefore b_{ef} = \rho b = 33.48
\]

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

\[
f_{cr} = 384
\]

\[
\lambda = \sqrt{\frac{500}{384}} = 1.20
\]

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

\[
\therefore b_{ef} = .672 \times 52 = 35
\]
(iii) AREAS - GROSS = \((36 + 36 + 64)\) \(\times 1.2 =
\]
\(A_g = 163\, \text{mm}^2\)
\(Effect = (33.48 + 33.48 + 35)\) \(\times 1.2 =
\]
\(A_E = 122\, \text{mm}^2\)

(B) TENSION CAPACITY – (AS4600 Section 3.2)

\[N^* \leq \phi A_g f_y \quad \text{or} \quad \phi \left(0.85 k_r A_u f_u\right)\]
\[\phi = 0.9 \quad A_g = 163 \quad A_u = 163 - \left(18 \times 1.2\right) = 141\, \text{mm}^2\]
\[f_y = 500 \quad f_u = 550 \quad k_r = 1.0\]
\[N^*_t = 0.9 \times 85 \times 122 \times 550\]
\[= 51.3\]
\[\text{or}\]
\[N^*_t = 0.9 \times 163 \times 500 = 73.35\]

\[N^*_t \leq 51.3\, \text{kN}\]

(C) COMPRESSION CAPACITY - (AS/NZS4600 Section 3.4)

\[N^* \leq \phi e N_s \quad \text{or} \quad \phi c N_c\]

(i) \(\phi e = 0.85 \quad N_s = A_u f_y = 141 \times 500\]
\[\Rightarrow \phi e N_s = 0.85 \times 141 \times 500\]
\[= 59.92\, \text{kN}\]

(ii) Determine Section Properties (Full)

\[I_{xx} = 2\left(36 \times 1.2 \times 31.5^2\right) + \frac{62^3 \times 1.2}{12}\]
\[= 109.56 \times 10^3 \, \text{mm}^4\]
\[A = 163 \Rightarrow r_{xx} = \sqrt{\frac{10956}{163}} = 25.9\]

(iii) Determine \(\lambda_c\) (Section 3.4.2)

\[f_{\infty} = \frac{\pi^2 E}{\left(\frac{2700}{25.9}\right)^2}\]
\[= 182\, \text{MPa}\]
Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:
Cyclones, Earthquakes, Fire, Termites, Tornadoes

\[ \lambda_c = \frac{500}{182} = 1.657 \quad \text{Eqn 3.4.1(5)} \]
\[ f_u = \left( \frac{877}{A^2} \right) f_y \quad \text{Eqn 3.4.1(3)} \]

\[ = 0.319 \times 500 = 159 \]
\[ \Rightarrow \phi_c N_c = 0.85 \times 141 \times 159 \]
\[ = 19.06 kN \]

\[ \therefore N_c^* \leq 19.06 kN \quad \text{--- Per Stud} \]

**CHECK BOLT TEAR OUT** (Bearing) - (AS/NZS4660 Clause 5.3.4.2)

\[ V_b^* = \phi V_b \]
\[ \phi \approx C d \frac{f_u}{t} \]
\[ \phi = 0.6 \quad C = 4 - 0.1 \left( \frac{16}{1.2} \right) = 2.667 \]
\[ \approx \text{(Washers Built in) Table 5.3.4.2(A)} = 1.0 \]
\[ t = 1.2 \quad f_u = 550 \]
\[ \therefore V_b^* = 0.6 \times 2.667 \times 16 \times 1.2 \times 500 \]

\[ V_b^* \leq 16.99 kN \]

**SUMMARY PER STUD**
\[ N_c^* \leq 51.3 kN \]
\[ N_c^* \leq 59.9 kN \]
\[ V_b^* \leq 16.99 \]
Bottom Bracket

40x8 Material
\[ N^*_r \leq \phi A_y f_y \]
\[ \phi = 0.9 \quad A_y = 25 \times 8 = 200 \quad f_y = 300 \]
\[ = 54kN \]
\[ N^*_r \leq 54kN \]

Bolting to Bearer

2M10 Bolts
\[ N^*_f \leq \phi N^*_f \] (AS4100 9.3.2.2)
\[ \phi = 0.8 \quad f_{uf} = 400 \quad A_s = 60 \]
\[ N^*_f \leq 0.8 \times 400 \times 60 \]
\[ \leq 19.2kN / Bolt \Rightarrow 38.4kN \text{ for 2 Bolts} \]
\[ V^*_f \leq \phi V_f (\phi 62 f_{uf} k r(A_s n + n A_s)) \]
\[ \phi = 0.8 \quad f_{uf} = 400 \quad n = 1 \quad A_s = 78 \]
\[ n_s = 0 \]
\[ \Rightarrow V_f = 0.8 \times 62 \times 400 \times 78 \]
\[ = 15.5kN / Bolt \]
\[ \Rightarrow V^*_f \leq 31kN \]

Bottom Bracket \( \phi N^*_r = 38.4kN, \phi V_f = 31kN \)
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>
<thead>
<tr>
<th>Trial No.</th>
<th>Date Tested</th>
<th>Maximum Load Applied (kN)</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4 Aug 2011</td>
<td>55.9</td>
<td>Failure. Tear in wall panel stud section material. Loss of load carrying capacity. Bending of bearer top flange.</td>
</tr>
</tbody>
</table>

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 \( \Rightarrow \) tensile section capacity is 29.5 kN.

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

\[ \therefore \text{Section Uplift Capacity - 29.5kN} \]
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)

\[
\frac{N *}{\phi N_{u_r}} + \frac{V *}{\phi V_{u_r}} \leq 1.2 \quad FromBrochure
\]

\[
\phi N_{u_r} = \phi N_u \cdot X_{tc} \cdot X_{te} \cdot X_{ta}
\]
\[
\phi N_u = 22.3kN
\]
\[
X_{tc} = .89
\]
\[
X_{te} = 1.0
\]
\[
X_{ta} = N / A
\]
\[
\Rightarrow \phi N_{u_r} = .89 \times 22.3 = 19.85kN
\]

\[
\phi V_{u_r} = \phi V_u \cdot X_{sc} \cdot X_{se} \cdot X_{sa}
\]
\[
\phi V_u = 17.2kN
\]
\[
X_{sc} = .82
\]
\[
X_{se} = .36
\]
\[
sa = N / A
\]
\[
\therefore \phi V_{u_r} = .17.2 \times .82 \times .36 = 5.08kN
\]

\[
\therefore \frac{N *}{19.85} + \frac{V *}{5.08} \leq 1.2 \quad \text{55mm Edge distance}
\]
b. M12 /N20 /125 – From Ramset Chem Std Limit State Capacities per Anchor (Tension and Shear)

\[ \phi N_{ur} = \phi N_u \cdot X_{tc} \cdot X_{te} \cdot X_{ta} \]
\[ \phi N_{ur} = 22.3kN \]
\[ X_{tc} = .89 \]
\[ X_{te} = 1.0 \]
\[ X_{ta} = N / A \]
\[ \Rightarrow \phi N_{ur} = .89 \times 22.3 = 19.85kN \]

\[ \phi V_{ur} = \phi V_u \cdot X_{sc} \cdot X_{se} \cdot X_{sa} \]
\[ \phi V_{ur} = 17.2kN \]
\[ X_{sc} = .82 \]
\[ X_{se} = 1.0 \]
\[ sa = N / A \]
\[ \therefore \phi V_{ur} = .17.2 \times .82 \times 1 = 14.18kN \]

\[ \therefore \frac{N}{19.85} + \frac{V}{14.1} \leq 1.2 - 125\text{mm Edge distance} \]

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

\[ \phi N_{ur} = \phi N_u \cdot X_{tc} \cdot X_{te} \cdot X_{ta} \]
\[ \phi N_{ur} = 32.6kN \]
\[ X_{tc} = .95 \]
\[ X_{te} = 1.0 \]
\[ X_{ta} = N / A \]
\[ \Rightarrow \phi N_{ur} = .95 \times 32.6 = 30.97kN \]

\[ \phi V_{ur} = \phi V_u \cdot X_{sc} \cdot X_{se} \cdot X_{sa} \]
\[ \phi V_{ur} = 52.3 \]
\[ X_{sc} = .91 \]
\[ X_{se} = .58 \]
\[ sa = N / A \]
\[ \therefore \phi V_{ur} = 52.3 \times .91 \times .58 = 27.6kN \]

\[ \therefore \frac{N}{30.97} + \frac{V}{27.6} \leq 1.2 - 125\text{mm Edge distance M16} \]
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 $C_{pn} = 1.35$ and Roof $C_{pn} = 1.6$

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

   \[
   \frac{N^*}{\phi N_{ur}} + \frac{V^*}{\phi V_{ur}} \leq 1.2
   \]

   \[
   \therefore N^* \leq (1.2 - \frac{V^*}{\phi V_{ur}})\phi N_{ur}
   \]

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

   \[
   \therefore V^* = C_{pn} \frac{P_{anel Ht}}{2}
   \]

   \[
   = 1.35qz \times 1.5
   \]

   \[
   or = 2.025qz
   \]

   \[
   \therefore \text{THIS CAN BE TABULATED}
   \]

<table>
<thead>
<tr>
<th>NCC WIND CATEGORY</th>
<th>$qz$ kPa</th>
<th>$V^* = 2.025qz$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>1.5</td>
<td>3.038</td>
</tr>
<tr>
<td>C2</td>
<td>2.233</td>
<td>4.522</td>
</tr>
<tr>
<td>C3</td>
<td>3.286</td>
<td>6.654</td>
</tr>
<tr>
<td>C4</td>
<td>4.438</td>
<td>8.99</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>NZBC WIND CATEGORY</th>
<th>$qz$ kPa</th>
<th>$V^* = 2.025qz$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TC1</td>
<td>2.233</td>
<td>4.522</td>
</tr>
</tbody>
</table>

\[
N^* \leq (1.2 - \frac{V^*}{\phi V_{ur}})\phi N_{ur}
\]

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

   \[
   \therefore N^* \leq (1.2 - \frac{V^*}{5.08})19.5
   \]

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

   \[
   \therefore N^* \leq (1.2 - \frac{V^*}{14.1})19.5
   \]

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

   \[
   \therefore N^* \leq (1.2 - \frac{V^*}{27.6})30.97
   \]
Australian Owned and Made Modular Floor, Wall, Roof Truss System resistant to:
Cyclones, Earthquakes, Fire, Termites, Tornadoes

### NCC Wind Category

<table>
<thead>
<tr>
<th>CASE</th>
<th>NCC WIND CATEGORY</th>
<th>V* kN</th>
<th>MAX N* kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>C1</td>
<td>3.038</td>
<td>11.95</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>4.522</td>
<td>6.15</td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>6.554</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>8.99</td>
<td>N/A</td>
</tr>
<tr>
<td>b</td>
<td>C1</td>
<td>3.038</td>
<td>19.54</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>4.522</td>
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<tr>
<td></td>
<td>C3</td>
<td>6.654</td>
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<tr>
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<td>C4</td>
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</tr>
<tr>
<td>c</td>
<td>C1</td>
<td>3.038</td>
<td>33.75*</td>
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<td></td>
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<td>4.522</td>
<td>32.1*</td>
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<tr>
<td></td>
<td>C3</td>
<td>6.654</td>
<td>29.7*</td>
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<tr>
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<td>26.92</td>
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</table>

*Capacity to be limited to 29.5 kN as per load tests JCU-CTS*

### NZBC Wind Category

<table>
<thead>
<tr>
<th>CASE</th>
<th>NZBC WIND CATEGORY</th>
<th>V* kN</th>
<th>MAX N* kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>TC!</td>
<td>4.522</td>
<td>11.95</td>
</tr>
<tr>
<td>b</td>
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<tr>
<td>c</td>
<td>TC!</td>
<td>4.522</td>
<td>32.1</td>
</tr>
</tbody>
</table>

*Capacity to be limited to 29.5 kN as per load tests JCU-CTS*