

## COMPUTATIONS

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**Project:**

**Project No: 17457**

2130 Yarragon-Leongatha Road, Hallston  
VIC

**Client:**

David Trease Design for J. & L. Brogno

**Codes Used:**

AS1170 - Loading Code  
AS3600 - Concrete Structures  
AS4100 - Steel Structures

**Description:**

Design of building work relating to Structural matters

**Computations:**

Pages 1-39

**Date:**

20/12/2024

**INDUSTRIAL FLOOR SLABS V5.07**

Deery Consulting Structural Engineer

<b>Slab:</b>	<b>(Floor Slab SL01) 130mm thick slab, f'c = 32MPa, SL92 Top</b>	
<b>Subgrade:</b>	<b>Subgrade Modulus (K) = 40kPa/mm (5.4% CBR)</b>	
<b>Position:</b>	<b>Internal condition</b>	
<b>Post loads:</b>	<b>20kN Post / Point Load, incl. 1000kg Pallet on Floor (Standard Dexian)</b>	<b>OK (0.80)</b>
<b>Wheels:</b>	<b>Standard Forklift (3.5 Tonne) (Single Axle -2 wheels per axle)</b>	<b>OK (0.69)</b>
<b>UDL's:</b>	<b>30kPa floor UDL</b>	<b>OK (0.45)</b>

**Concrete Parameters**

Normal density concrete

Concrete strength (f'c) =	32 MPa	
Slab thickness (h) =	130 mm	
Concrete density (ρ) =	2400 kg/m <sup>3</sup>	Normal density = 2400kg/m <sup>3</sup>
fcmi =	35.3 MPa	AS 3600 - Table 3.1.2
Modulus of elasticity (Ec = ρ <sup>1.5</sup> *0.043*√fcmi) =	30024 MPa ± 20%	AS 3600 - Cl 3.1.2
Method for flexural strength =	C (C)&CA,(T)48,(A)S 3600,(O)ther	
90 day flexural tensile strength (f'ct.f = 0.438*(f'c) <sup>2/3</sup> ) =	4.415 MPa	
Section modulus (Z) =	2816667 mm <sup>3</sup>	
Poissons ratio (μ) =	0.15	
Radius of relative stiffness (l = [E*h <sup>3</sup> /(12*(1-μ <sup>2</sup> )*K)] <sup>1/4</sup> ) =	612 mm	
Point under consideration =	I (I)nternal, (C)orner, (E)dge	
Consider load transfer between slabs at a joint =	Y (Y)es, (N)o (Applicable - Corner and edge only)	
Corner transfer multiplier =	0.70 Chandler (Load transfer between slabs)	
Edge transfer multiplier =	0.85 Chandler (Load transfer between slabs)	
Consider distance reduction to edge of radius =	N (Y)es, (N)o	

**Concrete Abrasion Resistance**

Exposure classification = Exposure B1, 7 days curing under ambient conditions, 40mm cover - Table 1.7  
Abrasion = Medium or heavy pneumatic-tyred traffic (>3 t gross mass) - Table 1.6

**Soil Parameters**

Modulus of subgrade reaction (K) =	40 kPa/mm	
Geotechnical report =		
Recommend nominal subbase thickness (CBR=5.4) =	150 mm	T48-2009 - Table 1.5

**Conversions (Valid for CBR's between 2 & 30):**

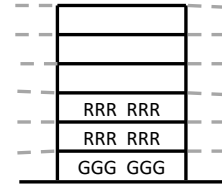
California Bearing Ratio (CBR) =	4 %	
Modulus of subgrade reaction (K) =	33.1 kPa/mm	
Modulus of subgrade reaction (K) =	40 kPa/mm	
California Bearing Ratio (CBR) =	5.4 %	
Bound sub-base thickness =	150 mm (100, 125, 150)	
California Bearing Ratio (CBR) =	4.0 % (max 12%)	
Equivalent design CBR =	25.4 % (35% max.)	T48-2009 - Figure 1.26
Modulus of subgrade reaction (K) =	76 kPa/mm	

**INDUSTRIAL FLOOR SLABS V5.07**

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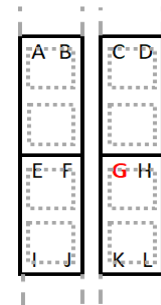
**Rack Loadings with Floor Pallets (Standard Dexian layout)** Note - Loaded areas overlap at mid-depth of slab when points closer than 249mm

Number of above ground rack levels =	2
Rack pallet weight =	1000 kg (2 pallets per rack)
Ground pallet weight =	1000 kg
Pallet side length =	1000 mm
Pallet side width =	1000 mm
Ground pallet load =	10.0 kPa
Working post load (RPL) =	20.0 kN
Aisle width =	1680 mm
Foot length (FL) =	140 mm
Foot width (Fw) =	80 mm
Foot area (Fa) =	11200 mm <sup>2</sup>
Radius of loaded area (r) =	60 mm
Equivalent radius of loaded area (b) =	63 mm
Material factor for racking (RK1) =	0.85 T48 - 0.75 to 0.85 - Table 1.16
Repetitions factor for racking (Rk2) =	1.00 T48 - Cl 3.3.6
Standard dexian racking system =	Y (Yes, (N)o



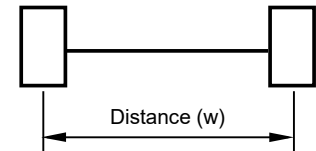
**Racking layout:**

Dist A - B =	838 mm
Dist A - E =	2591 mm
Dist B - C =	381 mm



**Wheel Loadings (Single Axle - 2 wheels per axle)**

Vehicle description =	Standard Forklift	
Maximum single axle load (P) =	3.50 Tonne	Axle = Approx. 2.45 * Capacity
Number of Wheels per axle =	2 No.	
Applied wheel load (unfactored) (WPL) =	1.75 Tonne	P=17.5kN
Distance between wheels (across axle) (w) =	2600 mm	
Distance between axles =	0 mm (0 for single axle)	
Tyre pressure (Pr) =	700 kPa	
Radius of loaded area (r) = $1000 \cdot \sqrt{WPL \cdot 10 / (\pi \cdot Pr)}$ =	89 mm	
Equivalent radius of loaded area (b) =	84 mm	
Material factor for wheel loads (Wk1) =	0.95	Chandler T48 - 0.85 to 0.95 - Table 1.16
Design life =	25 Years	
Daily cycles =	50 No.	
Number of repetitions in design life =	456250 No.	
Load repetition factor (Wk2) =	0.50	T48 - Table 1.17



**Uniform Floor Loading**

Number of pallets on the floor =	3	
Ground pallet weight =	1000 kg	
Pallet side length =	1000 mm	
Pallet end length =	1000 mm	
Ground Pallet load (q) =	30.0 kPa	
Aisle width =	2000 mm	
Material factor for UDL (Uk1) =	0.85	T48 - 0.75 to 0.85 - Table 1.16
Repetition factor for UDL (Uk2) =	0.75	T48 - Cl 3.3.6

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**POST FOOTING V5.03**

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**Geometry:** (Bored pier BP1) 600mm dia., 2300mm socket  
**Soil:**  $\gamma = 18\text{kN/m}^3$ ,  $\phi = 28^\circ$ , Cohesion = 70kPa  
**Resistance:** Bearing = 137kPa < 169kPa, Max. bearing = 270kPa < 348kPa

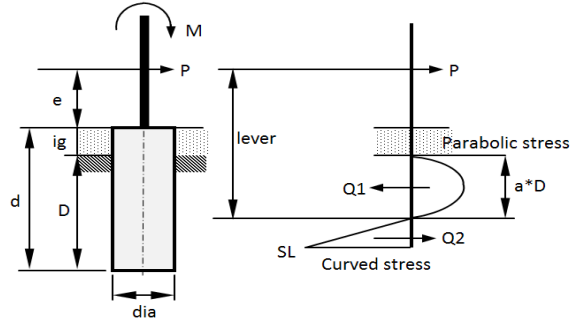
**OK (0.96)**

**Loadings & Footing Parameters - Minikin (For transient loadings only)**

Pier depth/diameter ratio = 3.8

Overturn. moment (M) = 38 kNm  
Lateral load (P) = 20 kN  
Load height (e) = mm  
Total footing depth (d) = 2300 mm  
Ignore top (ig) = 300 mm  
Diameter (dia) = 600 mm

Internal Friction ( $\phi$ ) = 28 ° ( $0^\circ \leq \phi \leq 45^\circ$ )  
Cohesion (C) = 70 kPa  
Soil weight ( $\gamma$ ) = 18.0 kN/m<sup>3</sup>  
 $N\phi = \tan^2(45^\circ + \phi/2) = 2.770$   
lever = 1769 mm  
Design depth (D=d-ig) = 2000 mm



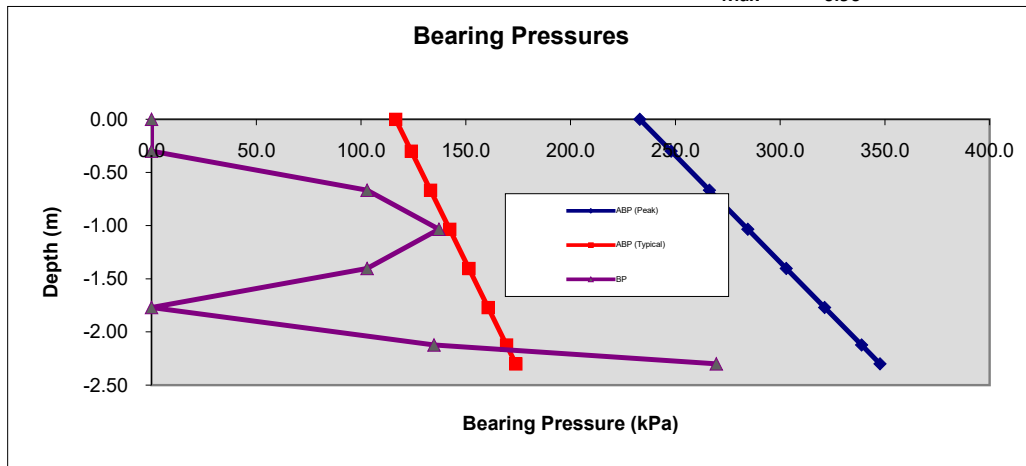
Note: Overturning moment (M) and Lateral load (P) \* Load height (e) are additive

**Allowable pressures** Error - Out-of-balance solution with Peq - Recommend solving for equilibrium

$n = (2*(e+ig)/D+1) = 1.300$   
 $a = 1-n/(1+3*n) = 0.735$   
 $Peq = (P*lever+Mo)/lever = 41.5 \text{ kN acting at } e = \text{mm (lever}=1769\text{mm)}$   
 $\text{Max BP (SL}=6*P*n/(D*dia)) = 269.6 \text{ kPa at } 2300 \text{ mm}$   
 $\text{Max BP (S1}=a^2/(4*(1-a)*SL)) = 137.1 \text{ kPa at } 1035 \text{ mm}$   
Force (Q1) = 80.6 kN at 1035 mm  
Force (Q2=Q1-P) = 60.6 kN at 2123 mm

**Allowable pressures**

Depth	Depth m	BP kPa	ABP kPa	ABP (FOS2) kPa	Ratio
Surface	0.00	0.0	233.0	116.5	0.00
ig	-0.30	0.0	248.0	124.0	0.00
ig + a/4*D	-0.67	102.8	266.3	133.1	0.77
ig + a/2*D	-1.03	137.1	284.6	142.3	0.96
ig + 3/4*a*D	-1.40	102.8	302.9	151.5	0.68
ig + a*D	-1.77	0.0	321.2	160.6	0.00
ig + a*D+2/3*(1-a)*D	-2.12	134.8	338.9	169.4	0.80
Bottom	-2.30	269.6	347.7	173.8	0.78
<b>Max =</b>					<b>0.96</b>



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**STEEL MEMBER V5.15**

Deery Consulting Structural Engineer

Doubly symmetric and compact

<b>Section:</b>	<b>(R1 - SG check LC11 +M02) 250UB25.7 (G300)</b>	
<b>Bending:</b>	<b><math>M_x^* = 18.0\text{kNm} &lt; \phi M_b(5300, \alpha_m=1.30) = 36.4\text{kNm}</math>, <math>\phi M_b(\alpha_m=1) = 28.0\text{kNm}</math></b>	<b>OK (0.49)</b>
	No minor bending	
<b>Shear:</b>	<b><math>V_x^* = 13.0\text{kN} &lt; \phi V_{vm} = 214.3\text{kN}</math> (Web area full depth)</b>	<b>OK (0.06)</b>
	No minor shear	
<b>Compression:</b>	No compression	
<b>Tension:</b>	<b><math>N_t^* = 16.0\text{kN} &lt; \phi N_t = 941.8\text{kN}</math></b>	<b>OK (0.02)</b>
<b>Combined:</b>	<b>In-plane = 0.20, Out-of-plane = 0.49</b>	<b>OK (0.49)</b>
<b>Torsion:</b>	No torsion	

**Bending & Shear - Section 5**

Max. restraint (2.5% flange force) = 1.9 kN

(M\* to include first order amplification as required - Cl 4.4.2)

Analysis values = M (M)annual, (L)eft, (P)osition (X) from analysis, (R)ight, (C)ritical

Major bending ( $M_x^*$ ) =	18.0 kNm	Major bending ( $M_x^*$ ) =	18 kNm
Minor bending ( $M_y^*$ ) =	0.000 kNm	Minor bending ( $M_y^*$ ) =	0 kNm
Shear Force ( $V_x^*$ ) =	13.0 kN	Torsion ( $M_z^*$ ) =	0 kNm
Shear Force ( $V_y^*$ ) =	0.000 kN	Shear Force ( $V_x^*$ ) =	13 kN
Effective length factor ( $k_e$ ) =	Calc	Shear Force ( $V_y^*$ ) =	0 kN
$k_e = (k_t=1.00) * (k_l=1.00) * (k_r=1.00) =$	1.00 (From Le Tab)	Span / Segment Length (L) =	5300 mm
Effective length ( $L_e = L * k_e$ ) =	5300 mm	Moment modification factor ( $\alpha_m$ ) =	1.30
$\phi =$	0.9 Table 3.4		
$\phi M_{sx} =$	91.9 kNm	Bending (x) =	OK (0.49)
$\phi M_{bx}(\alpha_m=1) =$	28.0 kNm		
$\phi M_{bx} =$	36.4 kNm	Shear =	OK (0.06)
$\phi M_{sy} =$	17.8 kNm		
$\phi V_v =$	214.3 kN		
$\phi V_{vm} =$	214.3 kN	$I_x =$	35.4 x10 <sup>6</sup> mm <sup>4</sup>
$\phi M_z =$	1.46 kNm	S.Wt =	0.257 kN/m

**Compression - Section 6 (No compression)**

Axial compression ( $N_c^*$ ) =	0.0 kN	Axial compression ( $N_c^*$ ) =	0 kN
Major axis length ( $L_x$ ) =	5300 mm	Eff. X length factor ( $k_{ex}$ ) =	1.00
Minor axis length ( $L_y$ ) =	1200 mm	Eff. Y length factor ( $k_{ey}$ ) =	1.00
Braced or Sway member =	S (B)raccd, (S)way		
		Major axis effective length ( $L_{ex}$ ) =	5300 mm
$\phi N_s =$	893.7 kN	Minor axis effective length ( $L_{ey}$ ) =	1200 mm
$\phi N_{cx}(k_{ex}=1.00) =$	741.5 kN		
In-Plane $\phi N_{cx}(k_{ex}=1.00) =$	741.5 kN		
$\phi N_{cy}(k_{ey}=1.00) =$	780.2 kN		
In-Plane $\phi N_{cy}(k_{ey}=1.00) =$	780.2 kN		
$\phi N_c =$	741.5 kN		

**Tension - Section 7**

Axial tension ( $N_t^*$ ) =	16.0 kN	Axial tension ( $N_t^*$ ) =	16 kN
$k_t =$	1.00 Table 7.3.2		
		Tension =	OK (0.02)
$\phi N_t =$	941.8 kN	Bending & Tension =	OK (0.48)

**Combined**

$\phi M_{rxt} =$	90.3 kNm	$\phi M_{ixt} =$	90.3 kNm	$\phi M_{oxt} =$	37.0 kNm
$\phi M_{rxc} =$	91.9 kNm	$\phi M_{ixc} =$	91.9 kNm	$\phi M_{oxc} =$	36.4 kNm
$\phi M_{ryt} =$	17.5 kNm	$\phi M_{iyc} =$	17.8 kNm	$\phi M_{tx} =$	37.0 kNm
$\phi M_{ryc} =$	17.8 kNm			$\phi M_{cx} =$	36.4 kNm

**R1 - SG check LC12 -M02**

**STEEL MEMBER V5.15**

Deery Consulting Structural Engineer

Doubly symmetric and compact

<b>Section:</b>	<b>(R1 - SG check LC12 -M02) 250UB25.7 (G300)</b>	
<b>Bending:</b>	<b><math>M_x^* = 26.0\text{kNm} &lt; \phi M_b(4300, \alpha_m=1.30) = 45.8\text{kNm}</math>, <math>\phi M_b(\alpha_m=1) = 35.2\text{kNm}</math></b>	<b>OK (0.57)</b>
	No minor bending	
<b>Shear:</b>	<b><math>V_x^* = 15.0\text{kN} &lt; \phi V_{vm} = 214.3\text{kN}</math> (Web area full depth)</b>	<b>OK (0.07)</b>
	No minor shear	
<b>Compression:</b>	<b><math>N_c^* = 24.0\text{kN} &lt; \phi N_c = 741.5\text{kN}</math></b>	<b>OK (0.03)</b>
<b>Combined:</b>	<b>In-plane = 0.32, Out-of-plane = 0.60</b>	<b>OK (0.60)</b>
<b>Tension:</b>	<b>No tension</b>	
<b>Torsion:</b>	<b>No torsion</b>	

**Bending & Shear - Section 5**

Max. restraint (2.5% flange force) = 2.7 kN

(M\* to include first order amplification as required - Cl 4.4.2)

Analysis values = M (M)annual, (L)eft, (P)osition (X) from analysis, (R)ight, (C)ritical

Major bending ( $M_x^*$ ) =	26.0 kNm	Major bending ( $M_x^*$ ) =	26 kNm
Minor bending ( $M_y^*$ ) =	0.000 kNm	Minor bending ( $M_y^*$ ) =	0 kNm
Shear Force ( $V_x^*$ ) =	15.0 kN	Torsion ( $M_z^*$ ) =	0 kNm
Shear Force ( $V_y^*$ ) =	0.000 kN	Shear Force ( $V_x^*$ ) =	15 kN
Effective length factor ( $k_e$ ) =	Calc	Shear Force ( $V_y^*$ ) =	0 kN
$k_e = (k_t=1.00) * (k_l=1.00) * (k_r=1.00) =$	1.00 (From Le Tab)	Span / Segment Length (L) =	4300 mm
Effective length ( $L_e = L * k_e$ ) =	4300 mm	Moment modification factor ( $\alpha_m$ ) =	1.30
$\phi =$	0.9 Table 3.4		
$\phi M_{sx} =$	91.9 kNm	Bending (x) =	OK (0.57)
$\phi M_{bx}(\alpha_m=1) =$	35.2 kNm		
$\phi M_{bx} =$	45.8 kNm	Shear =	OK (0.07)
$\phi M_{sy} =$	17.8 kNm		
$\phi V_v =$	214.3 kN		
$\phi V_{vm} =$	214.3 kN	$I_x =$	35.4 x10 <sup>6</sup> mm <sup>4</sup>
$\phi M_z =$	1.46 kNm	S.Wt =	0.257 kN/m

**Compression - Section 6**

Axial compression ( $N_c^*$ ) =	24.0 kN	Axial compression ( $N_c^*$ ) =	24 kN
Major axis length ( $L_x$ ) =	5300 mm	Eff. X length factor ( $k_{ex}$ ) =	1.00
Minor axis length ( $L_y$ ) =	1200 mm	Eff. Y length factor ( $k_{ey}$ ) =	1.00
Braced or Sway member =	S (B)raccd, (S)way		
$\phi N_s =$	893.7 kN	Major axis effective length ( $L_{ex}$ ) =	5300 mm
$\phi N_{cx}(k_{ex}=1.00) =$	741.5 kN	Minor axis effective length ( $L_{ey}$ ) =	1200 mm
In-Plane $\phi N_{cx}(k_{ex}=1.00) =$	741.5 kN	Compression =	OK (0.03)
$\phi N_{cy}(k_{ey}=1.00) =$	780.2 kN	Bending & Comp. =	OK (0.60)
In-Plane $\phi N_{cy}(k_{ey}=1.00) =$	780.2 kN		
$\phi N_c =$	741.5 kN		

**Tension - Section 7 (No tension)**

Axial tension ( $N_t^*$ ) =	0.0 kN	Axial tension ( $N_t^*$ ) =	kN
$k_t =$	1.00 Table 7.3.2		
$\phi N_t =$	941.8 kN		

**Combined**

$\phi M_{rxt} =$	91.9 kNm	$\phi M_{ixt} =$	91.9 kNm	$\phi M_{oxt} =$	45.8 kNm
$\phi M_{rxc} =$	89.4 kNm	$\phi M_{ixc} =$	88.9 kNm	$\phi M_{oxc} =$	44.4 kNm
$\phi M_{ryt} =$	17.8 kNm	$\phi M_{iyc} =$	17.2 kNm	$\phi M_{tx} =$	45.8 kNm
$\phi M_{ryc} =$	17.3 kNm			$\phi M_{cx} =$	44.4 kNm

**C1 - SG check LC12 +M02**

**STEEL MEMBER V5.15**

Deery Consulting Structural Engineer

Doubly symmetric and compact

<b>Section:</b>	<b>(C1 - SG check LC12 +M02) 250UB25.7 (G300)</b>	
<b>Bending:</b>	<b><math>M_x^* = 26.0\text{kNm} &lt; \phi M_b(2400, \alpha_m=1.30) = 78.8\text{kNm}</math>, <math>\phi M_b(\alpha_m=1) = 60.6\text{kNm}</math></b>	<b>OK (0.33)</b>
	No minor bending	
<b>Shear:</b>	<b><math>V_x^* = 18.0\text{kN} &lt; \phi V_{vm} = 214.3\text{kN}</math> (Web area full depth)</b>	<b>OK (0.08)</b>
	No minor shear	
<b>Compression:</b>	<b><math>N_c^* = 26.0\text{kN} &lt; \phi N_c = 503.3\text{kN}</math></b>	<b>OK (0.05)</b>
<b>Combined:</b>	<b>In-plane = 0.32, Out-of-plane = 0.36</b>	<b>OK (0.36)</b>
<b>Tension:</b>	<b>No tension</b>	
<b>Torsion:</b>	<b>No torsion</b>	

**Bending & Shear - Section 5**

Max. restraint (2.5% flange force) = 2.7 kN

(M\* to include first order amplification as required - Cl 4.4.2)

Analysis values = M (M)annual, (L)eft, (P)osition (X) from analysis, (R)ight, (C)ritical

Major bending ( $M_x^*$ ) =	26.0 kNm	Major bending ( $M_x^*$ ) =	26 kNm
Minor bending ( $M_y^*$ ) =	0.000 kNm	Minor bending ( $M_y^*$ ) =	0 kNm
Shear Force ( $V_x^*$ ) =	18.0 kN	Torsion ( $M_z^*$ ) =	0 kNm
Shear Force ( $V_y^*$ ) =	0.000 kN	Shear Force ( $V_x^*$ ) =	18 kN
Effective length factor ( $k_e$ ) =	Calc	Shear Force ( $V_y^*$ ) =	0 kN
$k_e = (k_t=1.00) * (k_l=1.00) * (k_r=1.00) =$	1.00 (From Le Tab)	Span / Segment Length (L) =	2400 mm
Effective length ( $L_e = L * k_e$ ) =	2400 mm	Moment modification factor ( $\alpha_m$ ) =	1.30
$\phi =$	0.9 Table 3.4		
$\phi M_{sx} =$	91.9 kNm	Bending ( $x$ ) =	OK (0.33)
$\phi M_{bx}(\alpha_m=1) =$	60.6 kNm		
$\phi M_{bx} =$	78.8 kNm	Shear =	OK (0.08)
$\phi M_{sy} =$	17.8 kNm		
$\phi V_v =$	214.3 kN		
$\phi V_{vm} =$	214.3 kN	$I_x =$	35.4 x10 <sup>6</sup> mm <sup>4</sup>
$\phi M_z =$	1.46 kNm	S.Wt =	0.257 kN/m

**Compression - Section 6**

Axial compression ( $N_c^*$ ) =	26.0 kN	Axial compression ( $N_c^*$ ) =	26 kN
Major axis length ( $L_x$ ) =	4150 mm	Eff. X length factor ( $k_{ex}$ ) =	2.20
Minor axis length ( $L_y$ ) =	1400 mm	Eff. Y length factor ( $k_{ey}$ ) =	1.00
Braced or Sway member =	S (B)raccd, (S)way		
$\phi N_s =$	893.7 kN	Major axis effective length ( $L_{ex}$ ) =	9130 mm
$\phi N_{cx}(k_{ex}=2.20) =$	503.3 kN	Minor axis effective length ( $L_{ey}$ ) =	1400 mm
In-Plane $\phi N_{cx}(k_{ex}=1.00) =$	793.7 kN	Compression =	OK (0.05)
$\phi N_{cy}(k_{ey}=1.00) =$	745.7 kN	Bending & Comp. =	OK (0.36)
In-Plane $\phi N_{cy}(k_{ey}=1.00) =$	745.7 kN		
$\phi N_c =$	503.3 kN		

**Tension - Section 7 (No tension)**

Axial tension ( $N_t^*$ ) =	0.0 kN	Axial tension ( $N_t^*$ ) =	kN
$k_t =$	1.00 Table 7.3.2		
$\phi N_t =$	941.8 kN		

**Combined**

$\phi M_{rxt} =$	91.9 kNm	$\phi M_{ixt} =$	91.9 kNm	$\phi M_{oxt} =$	78.8 kNm
$\phi M_{rxc} =$	89.2 kNm	$\phi M_{ixc} =$	88.9 kNm	$\phi M_{oxc} =$	76.0 kNm
$\phi M_{ryt} =$	17.8 kNm	$\phi M_{iyc} =$	17.2 kNm	$\phi M_{tx} =$	78.8 kNm
$\phi M_{ryc} =$	17.3 kNm			$\phi M_{cx} =$	76.0 kNm

**C1 - SG check LC12 -M02**

**STEEL MEMBER V5.15**

Deery Consulting Structural Engineer

Doubly symmetric and compact

<b>Section:</b>	<b>(C1 - SG check LC12 -M02) 250UB25.7 (G300)</b>	
<b>Bending:</b>	<b><math>M_x^* = 39.0\text{kNm} &lt; \phi M_b(2400, \alpha_m=1.30) = 78.8\text{kNm}</math>, <math>\phi M_b(\alpha_m=1) = 60.6\text{kNm}</math></b>	<b>OK (0.50)</b>
	No minor bending	
<b>Shear:</b>	<b><math>V_x^* = 18.0\text{kN} &lt; \phi V_{vm} = 214.3\text{kN}</math> (Web area full depth)</b>	<b>OK (0.08)</b>
	No minor shear	
<b>Compression:</b>	<b><math>N_c^* = 26.0\text{kN} &lt; \phi N_c = 503.3\text{kN}</math></b>	<b>OK (0.05)</b>
<b>Combined:</b>	<b>In-plane = 0.46, Out-of-plane = 0.53</b>	<b>OK (0.53)</b>
<b>Tension:</b>	<b>No tension</b>	
<b>Torsion:</b>	<b>No torsion</b>	

**Bending & Shear - Section 5**

Max. restraint (2.5% flange force) = 4.1 kN

(M\* to include first order amplification as required - Cl 4.4.2)

Analysis values = M (M)annual, (L)eft, (P)osition (X) from analysis, (R)ight, (C)ritical

Major bending ( $M_x^*$ ) =	39.0 kNm	Major bending ( $M_x^*$ ) =	39 kNm
Minor bending ( $M_y^*$ ) =	0.000 kNm	Minor bending ( $M_y^*$ ) =	0 kNm
Shear Force ( $V_x^*$ ) =	18.0 kN	Torsion ( $M_z^*$ ) =	0 kNm
Shear Force ( $V_y^*$ ) =	0.000 kN	Shear Force ( $V_x^*$ ) =	18 kN
Effective length factor ( $k_e$ ) =	Calc	Shear Force ( $V_y^*$ ) =	0 kN
$k_e = (k_t=1.00) * (k_l=1.00) * (k_r=1.00) =$	1.00 (From Le Tab)	Span / Segment Length (L) =	2400 mm
Effective length ( $L_e = L * k_e$ ) =	2400 mm	Moment modification factor ( $\alpha_m$ ) =	1.30
$\phi =$	0.9 Table 3.4		
$\phi M_{sx} =$	91.9 kNm	Bending ( $x$ ) =	OK (0.50)
$\phi M_{bx}(\alpha_m=1) =$	60.6 kNm		
$\phi M_{bx} =$	78.8 kNm	Shear =	OK (0.08)
$\phi M_{sy} =$	17.8 kNm		
$\phi V_v =$	214.3 kN		
$\phi V_{vm} =$	214.3 kN	$I_x =$	35.4 x10 <sup>6</sup> mm <sup>4</sup>
$\phi M_z =$	1.46 kNm	S.Wt =	0.257 kN/m

**Compression - Section 6**

Axial compression ( $N_c^*$ ) =	26.0 kN	Axial compression ( $N_c^*$ ) =	26 kN
Major axis length ( $L_x$ ) =	4150 mm	Eff. X length factor ( $k_{ex}$ ) =	2.20
Minor axis length ( $L_y$ ) =	1400 mm	Eff. Y length factor ( $k_{ey}$ ) =	1.00
Braced or Sway member =	S (B)raccd, (S)way		
$\phi N_s =$	893.7 kN	Major axis effective length ( $L_{ex}$ ) =	9130 mm
$\phi N_{cx}(k_{ex}=2.20) =$	503.3 kN	Minor axis effective length ( $L_{ey}$ ) =	1400 mm
In-Plane $\phi N_{cx}(k_{ex}=1.00) =$	793.7 kN	Compression =	OK (0.05)
$\phi N_{cy}(k_{ey}=1.00) =$	745.7 kN	Bending & Comp. =	OK (0.53)
In-Plane $\phi N_{cy}(k_{ey}=1.00) =$	745.7 kN		
$\phi N_c =$	503.3 kN		

**Tension - Section 7 (No tension)**

Axial tension ( $N_t^*$ ) =	0.0 kN	Axial tension ( $N_t^*$ ) =	kN
$k_t =$	1.00 Table 7.3.2		
$\phi N_t =$	941.8 kN		

**Combined**

$\phi M_{rxt} =$	91.9 kNm	$\phi M_{ixt} =$	91.9 kNm	$\phi M_{oxt} =$	78.8 kNm
$\phi M_{rxc} =$	89.2 kNm	$\phi M_{ixc} =$	88.9 kNm	$\phi M_{oxc} =$	76.0 kNm
$\phi M_{ryt} =$	17.8 kNm	$\phi M_{iyc} =$	17.2 kNm	$\phi M_{tx} =$	78.8 kNm
$\phi M_{ryc} =$	17.3 kNm			$\phi M_{cx} =$	76.0 kNm

**STEEL MULLION V5.04**

Deery Consulting Structural Engineer

<b>Member:</b>	(Mullion M1) 180UB22.2 (G300) - No flybracing	
<b>Bending:</b>	M.in* = 12.4kNm < $\phi Mb(1400, \alpha m=1.00) = 45.0kNm$	OK (0.27)
	M.out* = 13.7kNm < $\phi Mb(6200, \alpha m=1.13) = 17.7kNm$	OK (0.77)
<b>Combined:</b>	In-plane = 0.29, Out-of-plane = 0.81	OK (0.81)
<b>Deflection:</b>	$\delta W L s . i n = L / 563$ (11mm), $\delta W L s . o u t = L / 597$ (10mm)	OK
<b>Reactions:</b>	(Each end) R.in* = 8.0kN, (Each end) R.out* = 7.5kN	

**Geometry**

Length (L=Lex) =	6200 mm	Use for Ley =	G (G)irts,(F)lybraces,(C)ustom
Centres (cts) =	2200 mm		
Girt centres = eff. length (Le) =	1400 mm (Outside flange)	Effective length (Ley) =	1400 mm
Inward ( $\alpha m$ ) =	1.00	Eff. length inside (Leo) =	6200 mm
Inward ( $\alpha m$ ) =	1.00	No. Flybraces to inside flange =	> 500 for seg length)
		Outward ( $\alpha m . o$ ) =	1.13

**Loadings**

Wind area reduction not applied

Wall area =	13.6 m <sup>2</sup>	Apply wind reduction =	N (N)one,(S)ide,(W)ind,(L)ee
Ws/Wu =	0.68	Area reduction (ka) =	1.00 AS 1170.2 Table 5.4
Ult. wind load in (Wu.in) =	1.30 kPa		
cpe =	0.7	cpi =	0.2
		w.in* =	2.57 kN/m
Ult. wind load out (Wu.out) =	1.30 kPa		
cpe =	0.65	cpi =	0.2
		w.out* =	2.43 kN/m
Axial compression (Nc*) =	23.0 kN	Eccentricity (ecc) =	89.5 mm (+ve increases out)
		D/2 =	89.5 mm
<b>Horz. Point loads</b>			
Wind load (pwl.in*) =	kN	Position =	3100 mm from bottom
Wind load (pwl.out*) =	kN		
w.in* =	2.57 kN/m	w.out* =	2.43 kN/m
p.in* =	0.00 kN	p.out* =	0.00 kN
M.in* =	12.4 kNm	M.out* =	11.7 kNm
M.(in+e)* =	12.4 kNm	M.(out+e)* =	13.7 kNm
		R.in* =	8.0 kN
		R.out* =	7.5 kN
		M.e* =	2.06 kNm
		M.(crit+e)* =	13.7 kNm

**Capacity**

Description =	180UB22.2 (G300)	Warping constant (Iw) =	8.71 x10 <sup>9</sup> mm <sup>6</sup>
Flange yield (fyf) =	320 MPa	Torsional constant (J) =	81.6 x10 <sup>3</sup> mm <sup>4</sup>
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	195 x10 <sup>3</sup> mm <sup>3</sup>
Area (Ag=An) =	2820 mm <sup>2</sup>	Effective section mod. (Zey) =	40.7 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Ix) =	15.3 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4
Stiffness (Iy) =	1.22 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4
ab =	0 (comp)	rx =	73.7 mm
$\phi$ =	0.9 Table 3.4	ry =	20.8 mm
<b>Bending</b>		$\phi M s y = \phi * \min (f y f , f y w ) * Z e y =$	11.7 kNm - Cl 5.2.1
Msx = min(fyf,fyw)*Zex =	62.4 kNm - Cl 5.2.1	$\phi M s x =$	56.2 kNm
Inward: Moa =	137.1 kNm $\alpha s = 0.801$	$\phi M b x =$	45.0 kNm
Outward: Moa =	20.9 kNm $\alpha s = 0.280$	$\phi M b x . o =$	17.7 kNm < Critical
<b>Compression</b>		$\phi N s = \phi * k f * A n * f y =$	812.2 kN
$\alpha c x =$	0.573	$\phi N c x = \phi N c x 1 = \alpha c x * \phi N s =$	465.7 kN
$\alpha c y =$	0.707	$\phi N c y = \phi N c y 1 = \alpha c y * \phi N s =$	574.5 kN
Ley =	1400 mm	$\phi N c = \min (\phi N c x , \phi N c y ) =$	465.7 kN
<b>Combined</b>		$\phi M r x c = \phi M s x * ( 1 - N c * / \phi N s ) =$	54.6 kNm OK (0.25)
		$\phi M i x c = \phi M s x * ( 1 - N c * / \phi N c x 1 ) =$	53.4 kNm OK (0.26)
		$\phi M o x c = \phi M b x . o * ( 1 - N c * / \phi N c y ) =$	17.0 kNm OK (0.81)
		In-plane member ratio = $M x * / \phi M s x + N c * / \phi N c x 1 =$	0.29 OK (0.29)
		Out-of-plane member ratio = $M x * / \phi M b x . o + N c * / \phi N c y =$	0.81 OK (0.81)

**Deflections**

Ireq'd WLs.in (L/250) =	6.8 x10 <sup>6</sup> mm <sup>4</sup>	< Critical	$\delta W L s . i n =$	11.0 mm	Span / 563
Ireq'd WLs.out (L/250) =	6.4 x10 <sup>6</sup> mm <sup>4</sup>		$\delta W L s . o u t =$	10.4 mm	Span / 597

**STEEL MULLION V5.04**

Deery Consulting Structural Engineer

<b>Member:</b>	(1Door mullion DM1) 180x75PFC (G300) - No flybracing	
<b>Bending:</b>	M.in* = 14.8kNm < $\phi Mb(1400, \alpha m=1.00) = 41.4\text{kNm}$	OK (0.36)
	M.out* = 13.9kNm < $\phi Mb(5850, \alpha m=1.13) = 20.1\text{kNm}$	OK (0.69)
<b>Combined:</b>	In-plane = 0.30, Out-of-plane = 0.69	OK (0.69)
<b>Deflection:</b>	$\delta W L s . i n = L / 461$ (13mm), $\delta W L s . o u t = L / 488$ (12mm)	OK
<b>Reactions:</b>	(Each end) R.in* = 10.1kN, (Each end) R.out* = 9.5kN	

**Geometry**

Length (L=Lex) =	5850 mm	Use for Ley =	G (G)irts,(F)lybraces,(C)ustom
Centres (cts) =	2950 mm		
Girt centres = eff. length (Le) =	1400 mm (Outside flange)	Effective length (Ley) =	1400 mm
Inward ( $\alpha m$ ) =	1.00	Eff. length inside (Leo) =	5850 mm
Inward ( $\alpha m$ ) =	1.00	No. Flybraces to inside flange =	> 500 for seg length)
		Outward ( $\alpha m . o$ ) =	1.13

**Loadings** Wind area reduction not applied

Wall area =	17.3 m <sup>2</sup>	Apply wind reduction =	N (N)one,(S)ide,(W)ind,(L)ee
Ws/Wu =	0.68	Area reduction (ka) =	1.00 AS 1170.2 Table 5.4
Ult. wind load in (Wu.in) =	1.30 kPa		
cpe =	0.7	cpi =	0.2
		w.in* =	3.45 kN/m
Ult. wind load out (Wu.out) =	1.30 kPa		
cpe =	0.65	cpi =	0.2
		w.out* =	3.26 kN/m
Axial compression (Nc*) =	0.0 kN	Eccentricity (ecc) =	90 mm (+ve increases out)
		D/2 =	90 mm
<b>Horz. Point loads</b>			
Wind load (pwl.in*) =	kN	Position =	2925 mm from bottom
Wind load (pwl.out*) =	kN		
w.in* =	3.45 kN/m	w.out* =	3.26 kN/m
p.in* =	0.00 kN	p.out* =	0.00 kN
M.in* =	14.8 kNm	M.out* =	13.9 kNm
M.(in+e)* =	14.8 kNm	M.(out+e)* =	13.9 kNm
		R.in* =	10.1 kN
		R.out* =	9.5 kN
		M.e* =	0.00 kNm
		M.(crit+e)* =	14.8 kNm

**Capacity**

Description =	180x75PFC (G300)	Warping constant (Iw) =	7.75 x10 <sup>9</sup> mm <sup>6</sup>	
Flange yield (fyf) =	300 MPa	Torsional constant (J) =	84.5 x10 <sup>3</sup> mm <sup>4</sup>	
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	182 x10 <sup>3</sup> mm <sup>3</sup>	
Area (Ag=An) =	2660 mm <sup>2</sup>	Effective section mod. (Zey) =	44.9 x10 <sup>3</sup> mm <sup>3</sup>	
Stiffness (Ix) =	14.1 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4	
Stiffness (Iy) =	1.51 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4	
ab =	0.5 (comp)	rx =	72.8 mm	
$\phi$ =	0.9 Table 3.4	ry =	23.8 mm	
<b>Bending</b>		$\phi M s y = \phi * \min (f y f , f y w ) * Z e y =$	12.1 kNm - Cl 5.2.1	
Msx = min(fyf,fyw)*Zex =	54.6 kNm - Cl 5.2.1	$\phi M s x =$	49.1 kNm	
Inward: Moa =	148.8 kNm $\alpha s = 0.842$	$\alpha m = 1.00$	$\phi M b x =$	41.4 kNm
Outward: Moa =	25.1 kNm $\alpha s = 0.363$	$\alpha m . o = 1.13$	$\phi M b x . o =$	20.1 kNm < Critical
<b>Compression</b>		$\phi N s = \phi * k f * A n * f y =$	718.2 kN	
$\alpha c x =$	0.559	$\phi N c x = \phi N c x 1 = \alpha c x * \phi N s =$	401.7 kN	
$\alpha c y =$	0.718	$\phi N c y = \phi N c y 1 = \alpha c y * \phi N s =$	515.5 kN	
Ley =	1400 mm	$\phi N c = \min (\phi N c x , \phi N c y ) =$	401.7 kN	
<b>Combined</b>		$\phi M r x c = \phi M s x * ( 1 - N c * / \phi N s ) =$	49.1 kNm OK (0.30)	
		$\phi M i x c = \phi M s x * ( 1 - N c * / \phi N c x 1 ) =$	49.1 kNm OK (0.30)	
		$\phi M o x c = \phi M b x . o * ( 1 - N c * / \phi N c y ) =$	20.1 kNm OK (0.69)	
		In-plane member ratio = $M x * / \phi M s x + N c * / \phi N c x 1 =$	0.30 OK (0.30)	
		Out-of-plane member ratio = $M x * / \phi M b x . o + N c * / \phi N c y =$	0.69 OK (0.69)	

**Deflections**

Ireq'd WLs.in (L/250) =	7.6 x10 <sup>6</sup> mm <sup>4</sup>	< Critical	$\delta W L s . i n =$	12.7 mm	Span / 461
Ireq'd WLs.out (L/250) =	7.2 x10 <sup>6</sup> mm <sup>4</sup>		$\delta W L s . o u t =$	12.0 mm	Span / 488

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**STEEL MULLION V5.04**

Deery Consulting Structural Engineer

<b>Member:</b>	(1Door mullion DM2) 150x75PFC (G300) - No flybracing	
<b>Bending:</b>	M.in* = 3.8kNm < $\phi Mb(1400, \alpha m=1.00)$ = 31.1kNm	OK (0.12)
	M.out* = 3.6kNm < $\phi Mb(3100, \alpha m=1.13)$ = 24.2kNm	OK (0.15)
<b>Combined:</b>	In-plane = 0.10, Out-of-plane = 0.15	OK (0.15)
<b>Deflection:</b>	$\delta W L s . i n = L / 2002$ (2mm), $\delta W L s . o u t = L / 2120$ (1mm)	OK
<b>Reactions:</b>	(Each end) R.in* = 4.9kN, (Each end) R.out* = 4.6kN	

**Geometry**

Length (L=Lex) =	3100 mm	Use for Ley =	G (G)irts,(F)lybraces,(C)ustom
Centres (cts) =	2700 mm		
Girt centres = eff. length (Le) =	1400 mm (Outside flange)	Effective length (Ley) =	1400 mm
Inward ( $\alpha m$ ) =	1.00	Eff. length inside (Leo) =	3100 mm
Inward ( $\alpha m$ ) =	1.00	No. Flybraces to inside flange =	> 500 for seg length)
		Outward ( $\alpha m . o$ ) =	1.13

**Loadings** Wind area reduction not applied

Wall area =	8.4 m <sup>2</sup>	Apply wind reduction =	N (N)one,(S)ide,(W)ind,(L)ee
Ws/Wu =	0.68	Area reduction (ka) =	1.00 AS 1170.2 Table 5.4
Ult. wind load in (Wu.in) =	1.30 kPa		
cpe =	0.7	cpi =	0.2
		w.in* =	3.16 kN/m
Ult. wind load out (Wu.out) =	1.30 kPa		
cpe =	0.65	cpi =	0.2
		w.out* =	2.98 kN/m
Axial compression (Nc*) =	0.0 kN	Eccentricity (ecc) =	75 mm (+ve increases out)
		D/2 =	75 mm
<b>Horz. Point loads</b>			
Wind load (pwl.in*) =	kN	Position =	1550 mm from bottom
Wind load (pwl.out*) =	kN		
w.in* =	3.16 kN/m	w.out* =	2.98 kN/m
p.in* =	0.00 kN	p.out* =	0.00 kN
M.in* =	3.8 kNm	M.out* =	3.6 kNm
M.(in+e)* =	3.8 kNm	M.(out+e)* =	3.6 kNm
		R.in* =	4.9 kN
		R.out* =	4.6 kN
		M.e* =	0.00 kNm
		M.(crit+e)* =	3.8 kNm

**Capacity**

Description =	150x75PFC (G300)	Warping constant (Iw) =	4.56 x10 <sup>9</sup> mm <sup>6</sup>
Flange yield (fyf) =	320 MPa	Torsional constant (J) =	56.6 x10 <sup>3</sup> mm <sup>4</sup>
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	129 x10 <sup>3</sup> mm <sup>3</sup>
Area (Ag=An) =	2250 mm <sup>2</sup>	Effective section mod. (Zey) =	38.5 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Ix) =	8.34 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4
Stiffness (Iy) =	1.29 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4
$\alpha b$ =	0.5 (comp)	rx =	60.9 mm
$\phi$ =	0.9 Table 3.4	ry =	23.9 mm
<b>Bending</b>		$\phi M s y = \phi * \min (f y f , f y w ) * Z e y =$	11.1 kNm - Cl 5.2.1
Msx = min(fyf,fyw)*Zex =	41.3 kNm - Cl 5.2.1	$\phi M s x =$	37.2 kNm
Inward: Moa =	108.9 kNm $\alpha s = 0.836$	$\phi M b x =$	31.1 kNm
Outward: Moa =	38.1 kNm $\alpha s = 0.575$	$\phi M b x . o =$	24.2 kNm < Critical
<b>Compression</b>		$\phi N s = \phi * k f * A n * f y =$	648.0 kN
$\alpha c x =$	0.761	$\phi N c x = \phi N c x 1 = \alpha c x * \phi N s =$	493.4 kN
$\alpha c y =$	0.706	$\phi N c y = \phi N c y 1 = \alpha c y * \phi N s =$	457.4 kN
Ley =	1400 mm	$\phi N c = \min (\phi N c x , \phi N c y ) =$	457.4 kN
<b>Combined</b>		$\phi M r x c = \phi M s x * ( 1 - N c * / \phi N s ) =$	37.2 kNm OK (0.10)
		$\phi M i x c = \phi M s x * ( 1 - N c * / \phi N c x 1 ) =$	37.2 kNm OK (0.10)
		$\phi M o x c = \phi M b x . o * ( 1 - N c * / \phi N c y ) =$	24.2 kNm OK (0.15)
		In-plane member ratio = Mx*/ $\phi M s x + N c * / \phi N c x 1 =$	0.10 OK (0.10)
		Out-of-plane member ratio = Mx*/ $\phi M b x . o + N c * / \phi N c y =$	0.15 OK (0.15)

**Deflections**

Ireq'd WLs.in (L/250) =	1.0 x10 <sup>6</sup> mm <sup>4</sup>	< Critical	$\delta W L s . i n =$	1.5 mm	Span / 2002
Ireq'd WLs.out (L/250) =	1.0 x10 <sup>6</sup> mm <sup>4</sup>		$\delta W L s . o u t =$	1.5 mm	Span / 2120

**STEEL WALL TIE V5.04**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Door header DH1) 150x75PFC (G300) (Web horizontal)</b>		
<b>Bending:</b>	<b>Mx.in* = 13.0kNm &lt; <math>\phi</math>Mbx.i(3450, <math>\alpha</math>m=1.13) = 22.6kNm</b>		<b>OK (0.58)</b>
	<b>Mx.out* = 12.3kNm &lt; <math>\phi</math>Mbx.o(3450, <math>\alpha</math>m=1.30) = 25.9kNm</b>		<b>OK (0.47)</b>
	<b>My.vert* = 0.9kNm &lt; <math>\phi</math>Msy.d = 11.1kNm (Biaxial = 0.66)</b>		<b>OK (0.66)</b>
<b>Deflection:</b>	<b><math>\delta</math>in = L/335 (16mm), <math>\delta</math>out = L/355 (15mm), <math>\delta</math>v = L/712 (8mm)</b>		<b>Warning</b>
<b>H. Reactions:</b>	<b>(Each end) R*in = 9.6kN, R*out = 9.1kN</b>		
<b>V. Reactions:</b>	<b>(Each end) Rdl = 0.5kN, Rv* = 0.6kN</b>		

**Geometry**

Span (L) =	5400 mm	Horz. in segment length (Le) =	3450 mm
Wall loadwidth (wcts) =	3050 mm	$\alpha$ m =	1.13
Number of hangers =	(None)		
Web in horizontal dir. =	Y (Y)es,(N)o	Horz. out segment length (Leo) =	3450 mm
Vert. weak axis load dir. =	M Load (A),Load (B),(M)in	$\alpha$ mo =	1.30

**Loadings**

Ratio Ws/Wu =	0.68 (Refer wind analysis)	S.Wt =	0.177 kN/m
<b>Inward</b>			
Ult. inward wind load (Wu) =	1.30 kPa		
cpe =	0.7	cpi =	0.2
		w.in* =	3.57 kN/m
<b>Outward</b>			
Ult. outward wind load (Wu) =	1.30 kPa		
cpe =	0.65	cpi =	0.2
		w.out* =	3.37 kN/m
w.in* =	3.57 kN/m	w.out* =	3.37 kN/m
M.in* =	13.01 kNm	M.out* =	12.28 kNm
		R.in* =	9.6 kN
		R.out* =	9.1 kN
Mv* =	0.87 kNm (Self weight)		

**Capacity**

Description =	150x75PFC (G300)	Warping constant (Iw) =	4.56 x10 <sup>9</sup> mm <sup>6</sup>
Flange yield (fyf) =	320 MPa	Torsional constant (J) =	56.6 x10 <sup>3</sup> mm <sup>4</sup>
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	129 x10 <sup>3</sup> mm <sup>3</sup>
Area (Ag) =	2250 mm <sup>2</sup>	Effective section mod. (Zey) =	38.5 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Ix.in) =	8.34 x10 <sup>6</sup> mm <sup>4</sup>	Effective section mod. (ZeyR) =	38.5 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Iy.vert) =	1.29 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4
		Shear modulus (G) =	80000 MPa - Cl 1.4
$\phi$ =	0.9 Table 3.4	$\phi$ Msy.d =	11.1 kNm
Msx = min(fyf,fyw)*Zex =	41.3 kNm - Cl 5.2.1	$\phi$ Msx =	37.2 kNm
Inward: Moa =	33.6 kNm $\alpha$ s = 0.537	$\alpha$ m = 1.13	$\phi$ Mbx.i = 22.6 kNm
Outward: Moa =	33.6 kNm $\alpha$ s = 0.537	$\alpha$ m.o = 1.30	$\phi$ Mbx.o = 25.9 kNm
		Biaxial = max(Mx.in*/ $\phi$ Mbx.i & Mx.out*/ $\phi$ Mbx.o) + (My*/ $\phi$ Msy.d) =	0.66

**Deflections**

<b>Horizontal</b>			
Ireq'd WLS.in (L/360) =	9.0 x10 <sup>6</sup> mm <sup>4</sup>	< Critical	$\delta$ WLS.in = 16.1 mm
Ireq'd WLS.out (L/360) =	8.5 x10 <sup>6</sup> mm <sup>4</sup>		$\delta$ WLS.out = 15.2 mm
<b>Vertical</b>			
Ireq'd DL (L/360) =	0.7 x10 <sup>6</sup> mm <sup>4</sup>		$\delta$ Vert = 7.6 mm

**STEEL WALL TIE V5.04**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Door header DH2) 150x75PFC (G300) (Web horizontal)</b>		
<b>Bending:</b>	<b>Mx.in* = 8.6kNm &lt; <math>\phi</math>Mbx.i(3450, <math>\alpha</math>m=1.13) = 22.6kNm</b>		<b>OK (0.38)</b>
	<b>Mx.out* = 8.2kNm &lt; <math>\phi</math>Mbx.o(3450, <math>\alpha</math>m=1.30) = 25.9kNm</b>		<b>OK (0.31)</b>
	<b>My.vert* = 0.6kNm &lt; <math>\phi</math>Msy.d = 11.1kNm (Biaxial = 0.44)</b>		<b>OK (0.44)</b>
<b>Deflection:</b>	<b><math>\delta</math>in = L/620 (7mm), <math>\delta</math>out = L/656 (7mm), <math>\delta</math>v = L/1315 (3mm)</b>		<b>OK</b>
<b>H. Reactions:</b>	<b>(Each end) R*in = 7.9kN, R*out = 7.4kN</b>		
<b>V. Reactions:</b>	<b>(Each end) Rdl = 0.4kN, Rv* = 0.5kN</b>		

**Geometry**

Span (L) =	4400 mm	Horz. in segment length (Le) =	3450 mm
Wall loadwidth (wcts) =	3050 mm	$\alpha$ m =	1.13
Number of hangers =	(None)		
Web in horizontal dir. =	Y (Y)es,(N)o	Horz. out segment length (Leo) =	3450 mm
Vert. weak axis load dir. =	M Load (A),Load (B),(M)in	$\alpha$ mo =	1.30

**Loadings**

Ratio Ws/Wu =	0.68 (Refer wind analysis)	S.Wt =	0.177 kN/m
<b>Inward</b>			
Ult. inward wind load (Wu) =	1.30 kPa		
cpe =	0.7	cpi =	0.2
		w.in* =	3.57 kN/m
<b>Outward</b>			
Ult. outward wind load (Wu) =	1.30 kPa		
cpe =	0.65	cpi =	0.2
		w.out* =	3.37 kN/m
w.in* =	3.57 kN/m	w.out* =	3.37 kN/m
M.in* =	8.64 kNm	M.out* =	8.16 kNm
		R.in* =	7.9 kN
		R.out* =	7.4 kN
Mv* =	0.58 kNm (Self weight)		

**Capacity**

Description =	150x75PFC (G300)	Warping constant (Iw) =	4.56 x10 <sup>9</sup> mm <sup>6</sup>
Flange yield (fyf) =	320 MPa	Torsional constant (J) =	56.6 x10 <sup>3</sup> mm <sup>4</sup>
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	129 x10 <sup>3</sup> mm <sup>3</sup>
Area (Ag) =	2250 mm <sup>2</sup>	Effective section mod. (Zey) =	38.5 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Ix.in) =	8.34 x10 <sup>6</sup> mm <sup>4</sup>	Effective section mod. (ZeyR) =	38.5 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Iy.vert) =	1.29 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4
		Shear modulus (G) =	80000 MPa - Cl 1.4
$\phi$ =	0.9 Table 3.4	$\phi$ Msy.d =	11.1 kNm
Msx = min(fyf,fyw)*Zex =	41.3 kNm - Cl 5.2.1	$\phi$ Msx =	37.2 kNm
Inward: Moa =	33.6 kNm $\alpha$ s = 0.537	$\alpha$ m = 1.13	$\phi$ Mbx.i = 22.6 kNm
Outward: Moa =	33.6 kNm $\alpha$ s = 0.537	$\alpha$ m.o = 1.30	$\phi$ Mbx.o = 25.9 kNm
		Biaxial = max(Mx.in*/ $\phi$ Mbx.i & Mx.out*/ $\phi$ Mbx.o) + (My*/ $\phi$ Msy.d) =	0.44

**Deflections**

<b>Horizontal</b>			
Ireq'd WLS.in (L/360) =	4.8 x10 <sup>6</sup> mm <sup>4</sup>	< Critical	$\delta$ WLS.in = 7.1 mm
Ireq'd WLS.out (L/360) =	4.6 x10 <sup>6</sup> mm <sup>4</sup>		$\delta$ WLS.out = 6.7 mm
<b>Vertical</b>			
Ireq'd DL (L/360) =	0.4 x10 <sup>6</sup> mm <sup>4</sup>		$\delta$ Vert = 3.3 mm
			Span / 1315

**STEEL COLUMN V5.07**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Post P1) 89 x 5.0 SHS (G350)# (Connection 100mm from face)</b>	
<b>Compression:</b>	<b><math>N_c^* = 8.1\text{kN} &lt; \phi N_c = 214.7\text{kN}</math></b>	<b>OK (0.04)</b>
<b>Bending:</b>	<b><math>M_x^* = 1.2\text{kNm} &lt; \phi M_b(3600, \alpha_m=1.75) = 15.5\text{kNm}</math></b> <b>No minor bending</b>	<b>OK (0.08)</b>
<b>Combined:</b>	<b>In-plane = 0.11, Out-of-plane = 0.11</b>	<b>OK (0.11)</b>

**Geometry**

Column length (L) =	3600 mm	L = L <sub>b</sub> = L <sub>x</sub> = L <sub>y</sub> =	Y (Y)es,(N)o
<b>Compression:</b>			
Major axis length (L <sub>x</sub> ) =	3600 mm		
Minor axis length (L <sub>y</sub> ) =	3600 mm		
Effective length X factor (k <sub>ex</sub> ) =	1.00 Fig 4.6.3.2		
Effective length Y factor (k <sub>ey</sub> ) =	1.00 Fig 4.6.3.2	Holes =	0 mm <sup>2</sup> (axial reduction only)
<b>Bending:</b>			
Bending length (L <sub>b</sub> ) =	3600 mm	Eff. length L <sub>ex</sub> (3600,1.00) =	3600 mm
Bending effective length factor (k <sub>e</sub> ) =	1.00 Cl 5.6.3	Eff. length L <sub>ey</sub> (3600,1.00) =	3600 mm
Moment modification factor (α <sub>m</sub> ) =	1.75 Cl 5.6.1.1(a)	Eff. length L <sub>eb</sub> (3600,1.00) =	3600 mm

**Loadings**

S.Wt =	0.12 kN/m (excluded)		
Dead load comp. (N <sub>d</sub> ) =	4.0 kN	DL	LL
Live load comp. (N <sub>l</sub> ) =	2.3 kN	RB1	4.0 2.3 : L   L
N <sub>c</sub> * = 1.2*N <sub>d</sub> +1.5*N <sub>l</sub> =	8.1 kN	Σ	4.0 2.3
			Elastic Analysis
			type = 1 (1st order,(2)nd order
			type = B (B)raccd/(S)way - Cl 4.1.2
			β <sub>m</sub> = 0.0 cm = 0.60
			δ <sub>bx</sub> = cm/(1-N*/Nomx) = 1.00 and δ <sub>by</sub> = 1.00
Beam End Connection (For moment) =	F (C)ap,(F)ace,(M)anual,(L)oadng		
Major axis M <sub>x</sub> *=δ <sub>bx</sub> .N <sub>c</sub> *(D/2+100) =	1.2 kNm		
Minor axis M <sub>y</sub> *=δ <sub>by</sub> .M <sub>y</sub> * =	0.0 kNm - App. E	Minor bending (M <sub>y</sub> *) =	0.0 kNm
D =	89 mm (Bending about the strong axis only)		

**Capacity**

Section doubly symmetric			
Description = 89 x 5.0 SHS (G350)#	Warping constant (I <sub>w</sub> ) =	0 x10 <sup>9</sup> mm <sup>6</sup>	
Flange yield (f <sub>yf</sub> ) = 350 MPa	Torsional constant (J) =	3060 x10 <sup>3</sup> mm <sup>4</sup>	
Web yield (f <sub>yw</sub> ) = 350 MPa	Effective section mod. (Z <sub>ex</sub> ) =	49.1 x10 <sup>3</sup> mm <sup>3</sup>	
Area (A <sub>g</sub> ) = 1590 mm <sup>2</sup>	Effective section mod. (Z <sub>ey</sub> ) =	49.1 x10 <sup>3</sup> mm <sup>3</sup>	
Stiffness (I <sub>x</sub> ) = 1.82 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4	
Stiffness (I <sub>y</sub> ) = 1.82 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4	
r <sub>x</sub> = 33.8 mm	α <sub>b</sub> =	-0.5 (Comp.)	
r <sub>y</sub> = 33.8 mm	Net area (A <sub>n</sub> ) =	1590 mm <sup>2</sup>	
	Nom <sub>x</sub> = π <sup>2</sup> *E*I <sub>x</sub> /(k <sub>ex</sub> *L <sub>x</sub> ) <sup>2</sup> =	277 kN - Cl 4.6.2	
	Nom <sub>y</sub> = π <sup>2</sup> *E*I <sub>y</sub> /(k <sub>ey</sub> *L <sub>y</sub> ) <sup>2</sup> =	277 kN	
<b>Compression - Section 6</b>	φ <sub>Ns</sub> = φ*k <sub>f</sub> *A <sub>n</sub> *f <sub>y</sub> =	500.9 kN - Cl 6.2.1	
φ = 0.9 Table 3.4	φN <sub>c</sub> x = φN <sub>s</sub> *α <sub>c</sub> x =	214.7 kN - Cl 6.3.3	
α <sub>c</sub> x (L <sub>ex</sub> ) = 0.429	φN <sub>c</sub> y = φN <sub>s</sub> *α <sub>c</sub> y =	214.7 kN - Cl 6.3.3	
α <sub>c</sub> x1 (k <sub>ex</sub> =1, L <sub>ex</sub> =L <sub>x</sub> ) = 0.429	φN <sub>c</sub> =	214.7 kN	
α <sub>c</sub> y (L <sub>ey</sub> ) = 0.429	φN <sub>c</sub> x1 = φN <sub>s</sub> *α <sub>c</sub> x1 =	214.7 kN - Cl 8.4.2.2	
α <sub>c</sub> y1 (k <sub>ey</sub> =1, L <sub>ey</sub> =L <sub>y</sub> ) = 0.429	φN <sub>c</sub> y1 = φN <sub>s</sub> *α <sub>c</sub> y1 =	214.7 kN - Cl 8.4.2.2	
<b>Bending - Section 5</b>			
φ = 0.9 Table 3.4	φM <sub>s</sub> x =	15.5 kNm	φM <sub>s</sub> y = 15.5 kNm
M <sub>s</sub> x = min(f <sub>yf</sub> ,f <sub>yw</sub> )*Z <sub>ex</sub> = 17.2 kNm - Cl 5.2.1	φM <sub>s</sub> x =	15.5 kNm	φM <sub>b</sub> x = 15.5 kNm
M <sub>o</sub> a = 260.5 kNm α <sub>s</sub> = 1.000	α <sub>m</sub> = 1.75		φM <sub>b</sub> x(α <sub>m</sub> =1) = 15.5 kNm
<b>Combined - Section 8</b>			
Uniaxial bending about major axis - φM <sub>r</sub> x = φM <sub>s</sub> x*(1-N <sub>c</sub> */φN <sub>s</sub> ) =	15.2 kNm - Cl 8.3.2		OK (0.08)
In-plane capacity - φM <sub>i</sub> x = φM <sub>s</sub> x*(1-N <sub>c</sub> */φN <sub>c</sub> x1) =	14.9 kNm - Cl 8.4.2.2		OK (0.08)
Out-of-plane capacity - φM <sub>o</sub> x = φM <sub>b</sub> x*(1-N <sub>c</sub> */φN <sub>c</sub> y) =	14.9 kNm - Cl 8.4.4.1		OK (0.08)
In-plane member ratio = M <sub>x</sub> */φM <sub>s</sub> x + N <sub>c</sub> */φN <sub>c</sub> x1 =	0.11		OK (0.11)
In-plane member ratio = M <sub>y</sub> */φM <sub>s</sub> y + N <sub>c</sub> */φN <sub>c</sub> y1 =	0.04		OK (0.04)
Out-of-plane member ratio = M <sub>x</sub> */φM <sub>b</sub> x + N <sub>c</sub> */φN <sub>c</sub> y =	0.11		OK (0.11)
φM <sub>c</sub> x = min(φM <sub>o</sub> x & φM <sub>i</sub> x) =	14.9 kNm - Cl 8.4.5		
φM <sub>i</sub> y = φM <sub>s</sub> y*(1-N <sub>c</sub> */φN <sub>c</sub> y1) =	14.9 kNm - Cl 8.4.2.2		
Biaxial bending ratio = (M <sub>x</sub> */φM <sub>c</sub> x) <sup>1.4</sup> + (M <sub>y</sub> */φM <sub>i</sub> y) <sup>1.4</sup> =	0.03 Cl 8.4.5		OK (0.03)
Biaxial section ratio = N <sub>c</sub> */φN <sub>s</sub> + M <sub>x</sub> */φM <sub>s</sub> x + M <sub>y</sub> */φM <sub>s</sub> y =	0.09		OK (0.09)

**STEEL ROOF BEAM V5.12**

Deery Consulting Structural Engineer

<b>Member:</b>	(Roof beam RB1) 150x75PFC (G300) - No flybracing	
<b>Bending:</b>	M.dn*(max) = 8.1kNm < $\phi Mb(1500, \alpha.m.t=1.00)$ = 30.4kNm	OK (0.27)
	M.up*(max) = 8.8kNm < $\phi Mb(4000, \alpha.m.b=1.17)$ = 21.0kNm	OK (0.42)
<b>Shear:</b>	V.max*=8.1kN < $\phi Vvm = 155.5kN$ (Web area full depth)	OK (0.06)
	Vw* = 8.8kN < $\phi Vw = 155.5kN$ (Web area full depth)	OK (0.06)
<b>Deflection:</b>	$\delta DL = L/1013$ (4mm), $\delta \Psi s.LL = L/2542$ (2mm), $\delta WLs = L/478$ (8mm)	OK
<b>Precamber:</b>	Not required	
<b>Reactions:</b>	(Each end) Rdl = 4.0kN, Rll = 2.3kN, Rwl* = -12.3kN, R.dn* = 8.1kN, R.up* = 8.8kN	

**Geometry**

Span (L) =	4000 mm	Top flange restraint/purlin cts (Le) =	1500 mm (Top flange)
Centres (cts) =	4500 mm	Moment mod. factor ( $\alpha.m.t$ ) =	1.00
Design at =	M mm from LHS, (M)ax, (S)eg	Flybraces / Leb =	(≥ 500 for seg. length)
Span type =	S (S)ingle, (D)ouble		

Bottom $\alpha.m.b$ =	1.17 (Calc. $\alpha.m$ , Leb = 4000 mm)	Top $\alpha.m.t$ =	1.00 (Man. $\alpha.m$ , Le = 1500 mm)
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**Loadings**

Roof area (A) =	18.0 m <sup>2</sup>	Apply wind reduction =	Y (Y)es, (N)o
LL = 1.8/A+0.12 ≥ 0.25 =	0.25 kPa AS 1170.1 T3.2	Roof reduction (Ka) =	0.95 AS/NZS 1170.2, Table 5.4
		Ratio Ws/Wu =	0.68 (Refer wind analysis)

**Uniform dead loads**

Roof dead load (wdl) =	0.40 kPa *	4500 mm +	kN/m =	1.80 kN/m	
Other dead load (wdl) =	kPa *	4500 mm +	kN/m =	0.00 kN/m	
Down only load (wdl) =	kPa *	mm +	kN/m =	0.00 kN/m	
Include S.Wt =	Y (Y)es, (N)o		S.Wt =	0.18 kN/m	
		$\Sigma wdl.up$ =	1.98 kN/m	$\Sigma wdl$ =	1.98 kN/m

**Uniform live loads**

Roof live load (wll) =	0.25 kPa *	4500 mm +	kN/m =	1.13 kN/m	
Other live load (wll) =	kPa *	4500 mm +	kN/m =	0.00 kN/m	
Alternate point live load =	1.40 kN	Distr. to	1 members	$\Sigma wll$ =	1.13 kN/m

**Uniform wind loads**

Ult. wind load (Wu) =	1.30 kPa *	4500 mm			
Cp,e =	0.9	Cp,i =	0.2	w.wl* =	-6.15 kN/m (up)

**Point loads**

Dead load (pdl) =	kN	Position =	2000 mm from LHS
Live load (pll) =	kN	Shear using PL at support =	N (Y)es, (N)o
Wind load (pwl*) =	kN (-ve up)		
w* = 1.2*wdl + 1.5*wll =	4.06 kN/m	M* =	8.12 kNm (Max at 2000mm)
p* = 1.2*pdl + 1.5*pll =	0.00 kN	Mw.up* =	8.75 kNm (Max at 2000mm)
w.up* = 0.9*wdl.up + ww* =	4.38 kN/m (up)	V* =	8.12 kN
p.up* = 0.9*pdl + pwl* =	0.00 kN	Vw.up* =	8.75 kN

**Capacity**

Description =	150x75PFC (G300)	Warping constant (Iw) =	4.56 x10 <sup>9</sup> mm <sup>6</sup>	
Flange yield (fyf) =	320 MPa	Torsional constant (J) =	56.6 x10 <sup>3</sup> mm <sup>4</sup>	
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	129 x10 <sup>3</sup> mm <sup>3</sup>	
Area (Ag) =	2250 mm <sup>2</sup>	Effective section mod. (Zey) =	38.5 x10 <sup>3</sup> mm <sup>3</sup>	
Stiffness (Ix) =	8.34 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4	
$\phi$ =	0.9 Table 3.4	Shear modulus (G) =	80000 MPa - Cl 1.4	
Msx = min(fyf, fyw)*Zex =	41.3 kNm - Cl 5.2.1	$\phi$ Msx =	37.2 kNm	
Down: Moa =	98.2 kNm $\alpha.s.t = 0.817$	$\alpha.m.t = 1.00$	$\phi$ Mbx.d =	30.4 kNm
Uplift: Moa =	28.5 kNm $\alpha.s.b = 0.485$	$\alpha.m.b = 1.17$	$\phi$ Mbx.u =	21.0 kNm

**Deflections**

Ireq'd DL (L/250) =	2.1 x10 <sup>6</sup> mm <sup>4</sup>	$\delta DL$ =	4.0 mm	Span / 1013
Ireq'd LL (L/240) =	0.8 x10 <sup>6</sup> mm <sup>4</sup>	$\delta \Psi s.LL$ =	1.6 mm	Span / 2542
Ireq'd WLs (L/250) =	4.4 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\delta WLs$ =	8.4 mm	Span / 478

Max. precamber (0.3%*span) =	12 mm	Min. precamber =	15 mm
Precamber 80% of $\delta DL$ =	3 mm	Adopted precamber =	0 mm

**STEEL COLUMN V5.07**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Column C3_CENTRE) 100 x 6.0 SHS (G350)# (Connection 100mm from face)</b>	
<b>Compression:</b>	<b><math>N_c^* = 86.4\text{kN} &lt; \phi N_c = 289.9\text{kN}</math></b>	<b>OK (0.30)</b>
<b>Bending:</b>	<b><math>M_x^* = 13.0\text{kNm} &lt; \phi M_b(4000, \alpha_m=1.75) = 23.2\text{kNm}</math></b> <b>No minor bending</b>	<b>OK (0.56)</b>
<b>Combined:</b>	<b>In-plane = 0.86, Out-of-plane = 0.86</b>	<b>OK (0.86)</b>

**Geometry**

Column length (L) =	4000 mm	L = L <sub>b</sub> = L <sub>x</sub> = L <sub>y</sub> =	Y (Y)es,(N)o
<b>Compression:</b>			
Major axis length (L <sub>x</sub> ) =	4000 mm		
Minor axis length (L <sub>y</sub> ) =	4000 mm		
Effective length X factor (k <sub>ex</sub> ) =	1.00 Fig 4.6.3.2		
Effective length Y factor (k <sub>ey</sub> ) =	1.00 Fig 4.6.3.2	Holes =	0 mm <sup>2</sup> (axial reduction only)
<b>Bending:</b>			
Bending length (L <sub>b</sub> ) =	4000 mm	Eff. length L <sub>ex</sub> (4000,1.00) =	4000 mm
Bending effective length factor (k <sub>e</sub> ) =	1.00 Cl 5.6.3	Eff. length L <sub>ey</sub> (4000,1.00) =	4000 mm
Moment modification factor (α <sub>m</sub> ) =	1.75 Cl 5.6.1.1(a)	Eff. length L <sub>eb</sub> (4000,1.00) =	4000 mm

**Loadings**

S.Wt =	0.17 kN/m (excluded)		
Dead load comp. (N <sub>d</sub> ) =	9.2 kN	DL	LL
Live load comp. (N <sub>l</sub> ) =	50.2 kN	FB1	9.2 50.2 : L.R   L.R
N <sub>c</sub> * = 1.2*N <sub>d</sub> +1.5*N <sub>l</sub> =	86.4 kN	Σ	9.2 50.2
			Elastic Analysis
			1 (1st order,(2)nd order
			B (B)raced/(S)way - Cl 4.1.2
		β <sub>m</sub> =	0.0 cm = 0.60
		δ <sub>bx</sub> = cm/(1-N*/Nom <sub>x</sub> ) =	1.00 and δ <sub>by</sub> = 1.00
Beam End Connection (For moment) =	F (C)ap,(F)ace,(M)anual,(L)oadng		
Major axis M <sub>x</sub> *=δ <sub>bx</sub> .N <sub>c</sub> *(D/2+100) =	13.0 kNm		
Minor axis M <sub>y</sub> *=δ <sub>by</sub> .M <sub>y</sub> * =	0.0 kNm - App. E	Minor bending (M <sub>y</sub> *) =	0.0 kNm
D =	100 mm (Bending about the strong axis only)		

**Capacity**

<b>Section doubly symmetric</b>			
Description =	100 x 6.0 SHS (G350)#	Warping constant (I <sub>w</sub> ) =	0 x10 <sup>9</sup> mm <sup>6</sup>
Flange yield (f <sub>yf</sub> ) =	350 MPa	Torsional constant (J) =	5150 x10 <sup>3</sup> mm <sup>4</sup>
Web yield (f <sub>yw</sub> ) =	350 MPa	Effective section mod. (Z <sub>ex</sub> ) =	73.5 x10 <sup>3</sup> mm <sup>3</sup>
Area (A <sub>g</sub> ) =	2130 mm <sup>2</sup>	Effective section mod. (Z <sub>ey</sub> ) =	73.5 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (I <sub>x</sub> ) =	3.04 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4
Stiffness (I <sub>y</sub> ) =	3.04 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4
r <sub>x</sub> =	37.8 mm	α <sub>b</sub> =	-0.5 (Comp.)
r <sub>y</sub> =	37.8 mm	Net area (A <sub>n</sub> ) =	2130 mm <sup>2</sup>
<b>Compression - Section 6</b>		Nom <sub>x</sub> = π <sup>2</sup> *E*I <sub>x</sub> /(k <sub>ex</sub> *L <sub>x</sub> ) <sup>2</sup> =	375 kN - Cl 4.6.2
φ =	0.9 Table 3.4	Nom <sub>y</sub> = π <sup>2</sup> *E*I <sub>y</sub> /(k <sub>ey</sub> *L <sub>y</sub> ) <sup>2</sup> =	375 kN
α <sub>cx</sub> (L <sub>ex</sub> ) =	0.432	φN <sub>s</sub> = φ*k <sub>f</sub> *A <sub>n</sub> *f <sub>y</sub> =	671.0 kN - Cl 6.2.1
α <sub>cx1</sub> (k <sub>ex</sub> =1, L <sub>ex</sub> =L <sub>x</sub> ) =	0.432	φN <sub>cx</sub> = φN <sub>s</sub> *α <sub>cx</sub> =	289.9 kN - Cl 6.3.3
α <sub>cy</sub> (L <sub>ey</sub> ) =	0.432	φN <sub>cy</sub> = φN <sub>s</sub> *α <sub>cy</sub> =	289.9 kN - Cl 6.3.3
α <sub>cy1</sub> (k <sub>ey</sub> =1, L <sub>ey</sub> =L <sub>y</sub> ) =	0.432	φN <sub>c</sub> =	289.9 kN
		φN <sub>cx1</sub> = φN <sub>s</sub> *α <sub>cx1</sub> =	289.9 kN - Cl 8.4.2.2
		φN <sub>cy1</sub> = φN <sub>s</sub> *α <sub>cy1</sub> =	289.9 kN - Cl 8.4.2.2
<b>Bending - Section 5</b>			
φ =	0.9 Table 3.4	φM <sub>sy</sub> =	23.2 kNm
M <sub>sx</sub> = min(f <sub>yf</sub> ,f <sub>yw</sub> )*Z <sub>ex</sub> =	25.7 kNm - Cl 5.2.1	φM <sub>sx</sub> =	23.2 kNm
M <sub>oa</sub> =	393.1 kNm α <sub>s</sub> = 1.000	α <sub>m</sub> = 1.75	φM <sub>bx</sub> (α <sub>m</sub> =1) = 23.2 kNm
<b>Combined - Section 8</b>			
Uniaxial bending about major axis - φM <sub>rxc</sub> = φM <sub>sx</sub> *(1-N <sub>c</sub> */φN <sub>s</sub> ) =	20.2 kNm - Cl 8.3.2		OK (0.64)
In-plane capacity - φM <sub>ixc</sub> = φM <sub>sx</sub> *(1-N <sub>c</sub> */φN <sub>cx1</sub> ) =	16.3 kNm - Cl 8.4.2.2		OK (0.80)
Out-of-plane capacity - φM <sub>oxc</sub> = φM <sub>bx</sub> *(1-N <sub>c</sub> */φN <sub>cy</sub> ) =	16.3 kNm - Cl 8.4.4.1		OK (0.80)
In-plane member ratio = M <sub>x</sub> */φM <sub>sx</sub> + N <sub>c</sub> */φN <sub>cx1</sub> =	0.86		OK (0.86)
In-plane member ratio = M <sub>y</sub> */φM <sub>sy</sub> + N <sub>c</sub> */φN <sub>cy1</sub> =	0.30		OK (0.30)
Out-of-plane member ratio = M <sub>x</sub> */φM <sub>bx</sub> + N <sub>c</sub> */φN <sub>cy</sub> =	0.86		OK (0.86)
φM <sub>cx</sub> = min(φM <sub>oxc</sub> & φM <sub>ixc</sub> ) =	16.3 kNm - Cl 8.4.5		
φM <sub>iy</sub> = φM <sub>sy</sub> *(1-N <sub>c</sub> */φN <sub>cy1</sub> ) =	16.3 kNm - Cl 8.4.2.2		
Biaxial bending ratio = (M <sub>x</sub> */φM <sub>cx</sub> ) <sup>1.4</sup> + (M <sub>y</sub> */φM <sub>iy</sub> ) <sup>1.4</sup> =	0.73 Cl 8.4.5		OK (0.73)
Biaxial section ratio = N <sub>c</sub> */φN <sub>s</sub> + M <sub>x</sub> */φM <sub>sx</sub> + M <sub>y</sub> */φM <sub>sy</sub> =	0.69		OK (0.69)

**STEEL COLUMN V5.07**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Column C4 (for FB1 end)) 89 x 5.0 SHS (G350)# (Connection 100mm from face)</b>	
<b>Compression:</b>	<b><math>N_c^* = 43.2\text{kN} &lt; \phi N_c = 179.3\text{kN}</math></b>	<b>OK (0.24)</b>
<b>Bending:</b>	<b><math>M_x^* = 6.2\text{kNm} &lt; \phi M_b(4000, \alpha_m=1.75) = 15.5\text{kNm}</math></b> <b>No minor bending</b>	<b>OK (0.40)</b>
<b>Combined:</b>	<b>In-plane = 0.64, Out-of-plane = 0.64</b>	<b>OK (0.64)</b>

**Geometry**

Column length (L) =	4000 mm	L = L <sub>b</sub> = L <sub>x</sub> = L <sub>y</sub> =	Y (Y)es,(N)o
<b>Compression:</b>			
Major axis length (L <sub>x</sub> ) =	4000 mm		
Minor axis length (L <sub>y</sub> ) =	4000 mm		
Effective length X factor (k <sub>ex</sub> ) =	1.00 Fig 4.6.3.2		
Effective length Y factor (k <sub>ey</sub> ) =	1.00 Fig 4.6.3.2	Holes =	0 mm <sup>2</sup> (axial reduction only)
<b>Bending:</b>			
Bending length (L <sub>b</sub> ) =	4000 mm	Eff. length L <sub>ex</sub> (4000,1.00) =	4000 mm
Bending effective length factor (k <sub>e</sub> ) =	1.00 Cl 5.6.3	Eff. length L <sub>ey</sub> (4000,1.00) =	4000 mm
Moment modification factor (α <sub>m</sub> ) =	1.75 Cl 5.6.1.1(a)	Eff. length L <sub>eb</sub> (4000,1.00) =	4000 mm

**Loadings**

S.Wt =	0.12 kN/m (excluded)		
<b>Elastic Analysis</b>			
Dead load comp. (N <sub>d</sub> ) =	4.6 kN	DL	25.1
Live load comp. (N <sub>l</sub> ) =	25.1 kN	LL	25.1
N <sub>c</sub> * = 1.2*N <sub>d</sub> +1.5*N <sub>l</sub> =	43.2 kN	Σ	4.6
			25.1
		β <sub>m</sub> =	0.0
		δ <sub>bx</sub> = cm/(1-N*/Nom <sub>x</sub> ) =	1.00
			and δ <sub>by</sub> = 1.00
Beam End Connection (For moment) =	F (C)ap,(F)ace,(M)anual,(L)oadng		
Major axis M <sub>x</sub> *=δ <sub>bx</sub> .N <sub>c</sub> *(D/2+100) =	6.2 kNm		
Minor axis M <sub>y</sub> *=δ <sub>by</sub> .M <sub>y</sub> * =	0.0 kNm - App. E	Minor bending (M <sub>y</sub> *) =	0.0 kNm
D =	89 mm (Bending about the strong axis only)		

**Capacity**

<b>Section doubly symmetric</b>			
Description = 89 x 5.0 SHS (G350)#	Warping constant (I <sub>w</sub> ) =	0 x10 <sup>9</sup> mm <sup>6</sup>	
Flange yield (f <sub>yf</sub> ) = 350 MPa	Torsional constant (J) =	3060 x10 <sup>3</sup> mm <sup>4</sup>	
Web yield (f <sub>yw</sub> ) = 350 MPa	Effective section mod. (Z <sub>ex</sub> ) =	49.1 x10 <sup>3</sup> mm <sup>3</sup>	
Area (A <sub>g</sub> ) = 1590 mm <sup>2</sup>	Effective section mod. (Z <sub>ey</sub> ) =	49.1 x10 <sup>3</sup> mm <sup>3</sup>	
Stiffness (I <sub>x</sub> ) = 1.82 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4	
Stiffness (I <sub>y</sub> ) = 1.82 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4	
r <sub>x</sub> = 33.8 mm	α <sub>b</sub> =	-0.5 (Comp.)	
r <sub>y</sub> = 33.8 mm	Net area (A <sub>n</sub> ) =	1590 mm <sup>2</sup>	
	Nom <sub>x</sub> = π <sup>2</sup> *E*I <sub>x</sub> /(k <sub>ex</sub> *L <sub>x</sub> ) <sup>2</sup> =	225 kN - Cl 4.6.2	
	Nom <sub>y</sub> = π <sup>2</sup> *E*I <sub>y</sub> /(k <sub>ey</sub> *L <sub>y</sub> ) <sup>2</sup> =	225 kN	
<b>Compression - Section 6</b>	φ <sub>Ns</sub> = φ*k <sub>f</sub> *A <sub>n</sub> *f <sub>y</sub> =	500.9 kN - Cl 6.2.1	
φ = 0.9 Table 3.4	φN <sub>c</sub> x = φN <sub>s</sub> *α <sub>c</sub> x =	179.3 kN - Cl 6.3.3	
α <sub>c</sub> x (L <sub>ex</sub> ) = 0.358	φN <sub>c</sub> y = φN <sub>s</sub> *α <sub>c</sub> y =	179.3 kN - Cl 6.3.3	
α <sub>c</sub> x1 (k <sub>ex</sub> =1, L <sub>ex</sub> =L <sub>x</sub> ) = 0.358	φN <sub>c</sub> =	179.3 kN	
α <sub>c</sub> y (L <sub>ey</sub> ) = 0.358	φN <sub>c</sub> x1 = φN <sub>s</sub> *α <sub>c</sub> x1 =	179.3 kN - Cl 8.4.2.2	
α <sub>c</sub> y1 (k <sub>ey</sub> =1, L <sub>ey</sub> =L <sub>y</sub> ) = 0.358	φN <sub>c</sub> y1 = φN <sub>s</sub> *α <sub>c</sub> y1 =	179.3 kN - Cl 8.4.2.2	
<b>Bending - Section 5</b>			
φ = 0.9 Table 3.4	φM <sub>s</sub> x =	15.5 kNm	φM <sub>s</sub> y = 15.5 kNm
M <sub>s</sub> x = min(f <sub>yf</sub> ,f <sub>yw</sub> )*Z <sub>ex</sub> = 17.2 kNm - Cl 5.2.1	φM <sub>s</sub> x =	15.5 kNm	φM <sub>b</sub> x = 15.5 kNm
M <sub>o</sub> a = 234.4 kNm α <sub>s</sub> = 0.996	α <sub>m</sub> = 1.75		φM <sub>b</sub> x(α <sub>m</sub> =1) = 15.4 kNm
<b>Combined - Section 8</b>			
Uniaxial bending about major axis - φM <sub>r</sub> x = φM <sub>s</sub> x*(1-N <sub>c</sub> */φN <sub>s</sub> ) =	14.1 kNm - Cl 8.3.2		OK (0.44)
In-plane capacity - φM <sub>i</sub> x = φM <sub>s</sub> x*(1-N <sub>c</sub> */φN <sub>c</sub> x1) =	11.7 kNm - Cl 8.4.2.2		OK (0.53)
Out-of-plane capacity - φM <sub>o</sub> x = φM <sub>b</sub> x*(1-N <sub>c</sub> */φN <sub>c</sub> y) =	11.7 kNm - Cl 8.4.4.1		OK (0.53)
In-plane member ratio = M <sub>x</sub> */φM <sub>s</sub> x + N <sub>c</sub> */φN <sub>c</sub> x1 =	0.64		OK (0.64)
In-plane member ratio = M <sub>y</sub> */φM <sub>s</sub> y + N <sub>c</sub> */φN <sub>c</sub> y1 =	0.24		OK (0.24)
Out-of-plane member ratio = M <sub>x</sub> */φM <sub>b</sub> x + N <sub>c</sub> */φN <sub>c</sub> y =	0.64		OK (0.64)
φM <sub>c</sub> x = min(φM <sub>o</sub> x & φM <sub>i</sub> x) =	11.7 kNm - Cl 8.4.5		
φM <sub>i</sub> y = φM <sub>s</sub> y*(1-N <sub>c</sub> */φN <sub>c</sub> y1) =	11.7 kNm - Cl 8.4.2.2		
Biaxial bending ratio = (M <sub>x</sub> */φM <sub>c</sub> x) <sup>1.4</sup> + (M <sub>y</sub> */φM <sub>i</sub> y) <sup>1.4</sup> =	0.41 Cl 8.4.5		OK (0.41)
Biaxial section ratio = N <sub>c</sub> */φN <sub>s</sub> + M <sub>x</sub> */φM <sub>s</sub> x + M <sub>y</sub> */φM <sub>s</sub> y =	0.49		OK (0.49)

**STEEL COLUMN V5.07**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Column C4 (for FB2)) 89 x 5.0 SHS (G350)# (Cap plate connection)</b>	
<b>Compression:</b>	$Nc^* = 7.5kN < \phi Nc = 179.3kN$	<b>OK (0.04)</b>
<b>Bending:</b>	$Mx^* = 0.3kNm < \phi Mb(4000, \alpha m=1.75) = 15.5kNm$ No minor bending	<b>OK (0.02)</b>
<b>Combined:</b>	In-plane = 0.06, Out-of-plane = 0.06	<b>OK (0.06)</b>

**Geometry**

Column length (L) =	4000 mm	L = Lb = Lx = Ly =	Y (Yes,(N)o
<b>Compression:</b>			
Major axis length (Lx) =	4000 mm		
Minor axis length (Ly) =	4000 mm		
Effective length X factor (kex) =	1.00 Fig 4.6.3.2		
Effective length Y factor (key) =	1.00 Fig 4.6.3.2	Holes =	0 mm <sup>2</sup> (axial reduction only)
<b>Bending:</b>			
Bending length (Lb) =	4000 mm	Eff. length Lex(4000,1.00) =	4000 mm
Bending effective length factor (ke) =	1.00 Cl 5.6.3	Eff. length Ley(4000,1.00) =	4000 mm
Moment modification factor (αm) =	1.75 Cl 5.6.1.1(a)	Eff. length Leb(4000,1.00) =	4000 mm

**Loadings**

S.Wt =	0.12 kN/m (excluded)		
Dead load comp. (Ndl) =	0.6 kN	DL	LL
Live load comp. (Nll) =	4.5 kN	FB3	0.6 4.5 : L   L
Nc* = 1.2*Ndl+1.5*Nll =	7.5 kN	Σ	0.6 4.5
Beam End Connection (For moment) =	C (Cap),(Face),(Manual),(Loading)	Elastic Analysis	type = 1 (1st order,(2)nd order
Major axis Mx* = δbx.Nc*(D/2) =	0.3 kNm		type = B (B)raccd/(S)way - Cl 4.1.2
Minor axis My* = δby.My* =	0.0 kNm - App. E	βm =	0.0 cm = 0.60
D =	89 mm (Bending about the strong axis only)	δbx = cm/(1-N*/Nomx) =	1.00 and δby = 1.00

**Capacity**

Section doubly symmetric			
Description = 89 x 5.0 SHS (G350)#	Warping constant (Iw) =	0 x10 <sup>9</sup> mm <sup>6</sup>	
Flange yield (fyf) = 350 MPa	Torsional constant (J) =	3060 x10 <sup>3</sup> mm <sup>4</sup>	
Web yield (fyw) = 350 MPa	Effective section mod. (Zex) =	49.1 x10 <sup>3</sup> mm <sup>3</sup>	
Area (Ag) = 1590 mm <sup>2</sup>	Effective section mod. (Zey) =	49.1 x10 <sup>3</sup> mm <sup>3</sup>	
Stiffness (Ix) = 1.82 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4	
Stiffness (Iy) = 1.82 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4	
rx = 33.8 mm	ab =	-0.5 (Comp.)	
ry = 33.8 mm	Net area (An) =	1590 mm <sup>2</sup>	
<b>Compression - Section 6</b>		Nomx = π <sup>2</sup> *E*Ix/(kex*Lx) <sup>2</sup> =	225 kN - Cl 4.6.2
φ = 0.9 Table 3.4	φNs = φ*kf*An*fy =	500.9 kN - Cl 6.2.1	
αcx (kex=1, Lex=Lx) = 0.358	φNcx = φNs*αcx =	179.3 kN - Cl 6.3.3	
αcy (key=1, Ley=Ly) = 0.358	φNcy = φNs*αcy =	179.3 kN - Cl 6.3.3	
	φNc =	179.3 kN	
	φNcx1 = φNs*αcx1 =	179.3 kN - Cl 8.4.2.2	
	φNcy1 = φNs*αcy1 =	179.3 kN - Cl 8.4.2.2	
<b>Bending - Section 5</b>		φMsx =	15.5 kNm
Msx = min(fyf,fyw)*Zex =	17.2 kNm - Cl 5.2.1	φMbx =	15.5 kNm
Moa = 234.4 kNm	αs = 0.996	αm = 1.75	φMbx(αm=1) = 15.4 kNm
<b>Combined - Section 8</b>			
Uniaxial bending about major axis - φMrxc = φMsx*(1-Nc*/φNs) =	15.2 kNm - Cl 8.3.2		OK (0.02)
In-plane capacity - φMixc = φMsx*(1-Nc*/φNcx1) =	14.8 kNm - Cl 8.4.2.2		OK (0.02)
Out-of-plane capacity - φMoxc = φMbx*(1-Nc*/φNcy) =	14.8 kNm - Cl 8.4.4.1		OK (0.02)
In-plane member ratio = Mx*/φMsx + Nc*/φNcx1 =	0.06		OK (0.06)
In-plane member ratio = My*/φMsy + Nc*/φNcy1 =	0.04		OK (0.04)
Out-of-plane member ratio = Mx*/φMbx + Nc*/φNcy =	0.06		OK (0.06)
φMcx = min(φMoxc & φMixc) =	14.8 kNm - Cl 8.4.5		
φMiy = φMsy*(1-Nc*/φNcy1) =	14.8 kNm - Cl 8.4.2.2		
Biaxial bending ratio = (Mx*/φMcx) <sup>1.4</sup> + (My*/φMiy) <sup>1.4</sup> =	0.00 Cl 8.4.5		OK (0.00)
Biaxial section ratio = Nc*/φNs + Mx*/φMsx + My*/φMsy =	0.04		OK (0.04)

**STEEL MEMBER V5.15**

Deery Consulting Structural Engineer

Doubly symmetric and compact

**Section:** (Hanger H1) 89 x 5.0 SHS (G350)#  
**Bending:** No major bending  
No minor bending  
**Shear:** No major shear  
No minor shear  
**Compression:** No compression

**Tension:**  $N_t^* = 44.0\text{kN} < \phi N_t = 500.9\text{kN}$   
**Combined:** In-plane = 0.09, Out-of-plane = 0.00  
**Torsion:** No torsion

OK (0.09)  
OK (0.09)

**Bending & Shear - Section 5**

Max. restraint (2.5% flange force) = - kN

(M\* to include first order amplification as required - Cl 4.4.2)

Analysis values = M (M)annual, (L)eft, (P)osition (X) from analysis, (R)ight, (C)ritical

Major bending (Mx*) =	0.000 kNm	Major bending (Mx*) =	0.000 kNm
Minor bending (My*) =	0.000 kNm	Minor bending (My*) =	0 kNm
Shear Force (Vx*) =	0.000 kN	Torsion (Mz*) =	0 kNm
Shear Force (Vy*) =	0.000 kN	Shear Force (Vx*) =	0 kN
Effective length factor (ke) =	Calc	Shear Force (Vy*) =	0 kN
ke = (kt=1.00)*(kl=1.00)*(kr=1.00) =	1.00 (From Le Tab)	Span / Segment Length (L) =	1300 mm
Effective length (Le = L*ke) =	1300 mm	Moment modification factor ( $\alpha_m$ ) =	1.30
$\phi$ =	0.9 Table 3.4		
$\phi M_{sx}$ =	15.5 kNm		
$\phi M_{bx}(\alpha_m=1)$ =	15.5 kNm		
$\phi M_{bx}$ =	15.5 kNm		
$\phi M_{sy}$ =	15.5 kNm		
$\phi V_{vx}$ =	147.5 kN	$\phi V_{vy}$ =	147.5 kN
$\phi V_{vm}$ =	147.5 kN	lx =	1.82 x10 <sup>6</sup> mm <sup>4</sup>
$\phi M_z$ =	11.9 kNm	S.Wt =	0.125 kN/m

**Compression - Section 6 (No compression)**

Axial compression (Nc*) =	0.0 kN	Axial compression (Nc*) =	0 kN
Major axis length (Lx) =	1300 mm	Eff. X length factor (kex) =	1.00
Minor axis length (Ly) =	1300 mm	Eff. Y length factor (key) =	1.00
Braced or Sway member =	S (B)raced, (S)way		
		Major axis effective length (Lex) =	1300 mm
$\phi N_s$ =	500.9 kN	Minor axis effective length (Ley) =	1300 mm
$\phi N_{cx}(kex=1.00)$ =	461.8 kN		
In-Plane $\phi N_{cx}(kex=1.00)$ =	461.8 kN		
$\phi N_{cy}(key=1.00)$ =	461.8 kN		
In-Plane $\phi N_{cy}(key=1.00)$ =	461.8 kN		
$\phi N_c$ =	461.8 kN		

**Tension - Section 7**

Axial tension (Nt*) =	44.0 kN	Axial tension (Nt*) =	44 kN
kt =	1.00 Table 7.3.2		
		Tension = OK (0.09)	
$\phi N_t$ =	500.9 kN		

**Combined**

$\phi M_{rx}$ =	14.1 kNm	$\phi M_{ix}$ =	14.1 kNm	$\phi M_{ox}$ =	14.1 kNm
$\phi M_{rx}$ =	15.5 kNm	$\phi M_{ix}$ =	15.5 kNm	$\phi M_{ox}$ =	15.5 kNm
$\phi M_{ry}$ =	14.1 kNm	$\phi M_{iy}$ =	15.5 kNm	$\phi M_{tx}$ =	14.1 kNm
$\phi M_{ry}$ =	15.5 kNm			$\phi M_{cx}$ =	15.5 kNm

**STEEL FLOOR BEAM V5.07**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Floor Beam FB1) 300x90PFC (G300)</b>	
<b>Bending:</b>	<b>M*(max) = 66.9kNm &lt; <math>\phi</math>Mb(3100, <math>\alpha_m=1.30</math>) = 120.2kNm</b> <b>(Single span)</b>	<b>OK (0.56)</b>
<b>Shear:</b>	<b>V.max* = 43.2kN &lt; <math>\phi</math>Vvm = 414.7kN (Web area full depth)</b>	<b>OK (0.10)</b>
<b>Deflection:</b>	<b><math>\delta</math>DL = L/3149 (2mm), <math>\delta\psi_s.LL = L/823</math> (8mm), <math>\delta</math>Total = L/652 (10mm), 1kN mid. <math>\delta = 0.3</math>mm</b>	<b>OK</b>
<b>Precamber:</b>	<b>Not required</b>	
<b>Reactions:</b>	<b>(Each end) Rdl = 4.6kN, Rll = 25.1kN, R* = 43.2kN</b>	

**Geometry**

Span (L) =	6200 mm	Eff. Len. (Le) =	3100 mm
Centres (cts) =	2700 mm	$\alpha_m.t$ =	1.30
Design at =	M mm from LHS, (M)ax, (S)eg		
Span type =	S (S)ingle, (D)ouble		
Effective length (Le) =	3100 mm		
$\alpha_m$ =	1.30		

**Loadings**

Floor area =	16.7 m <sup>2</sup>	Live load type =	N (N)ormal, (S)torage, (M)annual
Apply reduction =	N (Y)es, (N)o		AS/NZS 1170.0 - Table 4.1
Floor reduction ( $\psi_a$ ) =	1.00 AS/NZS 1170.1 - Cl 3.4.2		

**Uniform dead loads**

Floor dead load (wdl) =	0.40 kPa *	2700 mm +	kN/m =	1.08 kN/m
Wall dead load (wdl) =	kPa *	2700 mm +	kN/m =	0.00 kN/m
Other dead load (wdl) =	kPa *	mm +	kN/m =	0.00 kN/m
Include S.Wt =	Y (Y)es, (N)o		S.Wt =	0.40 kN/m
			$\Sigma wdl$ =	1.48 kN/m

**Uniform live loads**

Floor live load (wll) =	3.00 kPa * $\psi_a$ *	2700 mm +	kN/m =	8.10 kN/m
Other live load (wll) =	kPa *	2700 mm +	kN/m =	0.00 kN/m
Alternate point live load =	4.50 kN	Distr. to	1 members	$\Sigma wll$ =
				8.10 kN/m

**Point loads**

Dead load (pdl) =	kN	Position =	3100 mm from LHS
Live load (pll) =	kN	Shear using PL at support =	N (Y)es, (N)o

Short term LL factor ( $\psi_{su}$ ) =	0.70	( $\psi_{sp}$ ) =	1.00
$w^* = 1.2 * wdl + 1.5 * wll$ =	13.93 kN/m	M* =	66.9 kNm (Max at 3100mm)
$p^* = 1.2 * pdl + 1.5 * pll$ =	0.00 kN	V* =	43.2 kN

**Capacity**

Description =	300x90PFC (G300)	Warping constant (Iw) =	58.1 x10 <sup>9</sup> mm <sup>6</sup>	
Flange yield (fyf) =	300 MPa	Torsional constant (J) =	304 x10 <sup>3</sup> mm <sup>4</sup>	
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	564 x10 <sup>3</sup> mm <sup>3</sup>	
Area (Ag) =	5110 mm <sup>2</sup>	Effective section mod. (Zey) =	82.3 x10 <sup>3</sup> mm <sup>3</sup>	
Stiffness (Ix) =	72.4 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4	
Stiffness (Iy) =	4.04 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4	
$\phi$ =	0.9 Table 3.4			
Msx = min(fyf, fyw) * Zex =	169.2 kNm - Cl 5.2.1	$\phi$ Msx =	152.3 kNm	
Positive: Moa.p =	173.4 kNm $\alpha_s.t = 0.607$	$\alpha_m.t = 1.30$	$\phi$ Msy =	22.2 kNm - Cl 5.2.1
Negative: Moa.n =	N.A.		$\phi$ Mbx.p =	120.2 kNm

**Deflections**

Ireq'd DL (L/360) =	8.3 x10 <sup>6</sup> mm <sup>4</sup>	$\delta$ DL =	2.0 mm	Span / 3149
Ireq'd $\psi_s.LL$ (L/360) =	31.7 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\psi_s.\delta LL$ =	7.5 mm	Span / 823
Ireq'd (DL+ $\psi_s.LL$ ) (L/250) =	27.7 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\delta$ Total =	9.5 mm	Span / 652
Max. precamber (0.3%*span) =	19 mm	Min. precamber =	15 mm	
Precamber 80% of $\delta$ DL =	2 mm	Adopted precamber =	0 mm	
		1kN midspan $\delta$ =	0.3 mm	

**STEEL FLOOR BEAM V5.07**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Floor Beam FB2) 200x75PFC (G300)</b>	
<b>Bending:</b>	<b>M*(max) = 21.7kNm &lt; <math>\phi</math>Mb(2700,<math>\alpha</math>m=1.30) = 48.1kNm</b> (Single span)	<b>OK (0.45)</b>
<b>Shear:</b>	<b>V.max*=17.4kN &lt; <math>\phi</math>Vvm = 207.4kN (Web area full depth)</b>	<b>OK (0.08)</b>
<b>Deflection:</b>	<b><math>\delta</math>DL = L/2704 (2mm), <math>\delta</math><math>\Psi</math>s.LL = L/696 (8mm), <math>\delta</math>Total = L/554 (10mm), 1kN mid. <math>\delta</math> = 0.9mm</b>	<b>OK</b>
<b>Precamber:</b>	<b>Not required</b>	
<b>Reactions:</b>	<b>(1 End) Rdl.max = 2.0kN, Rll.max = 10.0kN, R.max* = 17.4kN</b> <b>(1 End) Rdl.min = 1.7kN, Rll.min = 7.8kN, R.min* = 13.8kN</b>	

**Geometry**

Span (L) =	5400 mm	Eff. Len. (Le) =	2700 mm
Centres (cts) =	825 mm	$\alpha$ m.t =	1.30
Design at =	M mm from LHS, (M)ax, (S)eg		
Span type =	S (S)ingle,(D)ouble		
Effective length (Le) =	2700 mm		
$\alpha$ m =	1.30		

**Loadings**

Floor area =	4.5 m <sup>2</sup>	Live load type =	N (N)ormal, (S)torage, (M)annual
Apply reduction =	N (Y)es,(N)o		AS/NZS 1170.0 - Table 4.1
Floor reduction ( $\Psi$ a) =	1.00 AS/NZS 1170.1 - Cl 3.4.2		

**Uniform dead loads**

Floor dead load (wdl) =	0.40 kPa *	825 mm +	kN/m =	0.33 kN/m
Wall dead load (wdl) =	kPa *	825 mm +	kN/m =	0.00 kN/m
Other dead load (wdl) =	kPa *	mm +	kN/m =	0.00 kN/m
Include S.Wt =	Y (Y)es,(N)o		S.Wt =	0.23 kN/m
			$\Sigma$ wdl =	0.56 kN/m

**Uniform live loads**

Floor live load (wll) =	3.00 kPa * $\Psi$ a *	825 mm +	kN/m =	2.48 kN/m
Other live load (wll) =	kPa *	825 mm +	kN/m =	0.00 kN/m
Alternate point live load =	4.50 kN	Distr. to	1 members	$\Sigma$ wll =
				2.48 kN/m

**Point loads**

Dead load (pdl) =	0.6 kN	DL	LL	Position =	1400 mm from LHS
Live load (pll) =	4.5 kN	FB3	0.6	4.5 : L   L	
		$\Sigma$	0.6	4.5	using PL at support =
					N (Y)es,(N)o

Short term LL factor ( $\Psi$ su) =	0.70	( $\Psi$ sp) =	1.00
w* = 1.2*wdl + 1.5*wll =	4.38 kN/m	M* =	21.7 kNm (Max at 2257mm)
p* = 1.2*pdl + 1.5*pll =	7.49 kN	V* =	17.4 kN

**Capacity**

Description =	200x75PFC (G300)	Warping constant (Iw) =	10.5 x10 <sup>9</sup> mm <sup>6</sup>
Flange yield (fyf) =	300 MPa	Torsional constant (J) =	105 x10 <sup>3</sup> mm <sup>4</sup>
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	221 x10 <sup>3</sup> mm <sup>3</sup>
Area (Ag) =	2920 mm <sup>2</sup>	Effective section mod. (Zey) =	46.7 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Ix) =	19.1 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4
Stiffness (Iy) =	1.65 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4
$\phi$ =	0.9 Table 3.4		
Msx = min(fyf,fyw)*Zex =	66.3 kNm - Cl 5.2.1	$\phi$ Msx =	59.7 kNm
Positive: Moa.p =	70.9 kNm $\alpha$ s.t = 0.620	$\alpha$ m.t =	1.30
Negative: Moa.n =	N.A.	$\phi$ Msy =	12.6 kNm - Cl 5.2.1
		$\phi$ Mbx.p =	48.1 kNm

**Deflections** ( $\delta$   $\pm$ 2.5% with off-centre PL)

Ireq'd DL (L/360) =	2.5 x10 <sup>6</sup> mm <sup>4</sup>	$\delta$ DL =	2.0 mm	Span / 2704
Ireq'd $\Psi$ s.LL (L/360) =	9.9 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\Psi$ s. $\delta$ LL =	7.8 mm	Span / 696
Ireq'd (DL+ $\Psi$ s.LL) (L/250) =	8.6 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\delta$ Total =	9.8 mm	Span / 554
Max. precamber (0.3%*span) =	16 mm	Min. precamber =	15 mm	
Precamber 80% of $\delta$ DL =	2 mm	Adopted precamber =	0 mm	
		1kN midspan $\delta$ =	0.9 mm	

**STEEL FLOOR BEAM V5.07**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Floor Beam FB3) 200x75PFC (G300)</b>	
<b>Bending:</b>	<b>M*(max) = 2.2kNm &lt; <math>\phi</math>Mb(600, <math>\alpha_m=1.30</math>) = 59.7kNm</b> <b>(Single span)</b>	<b>OK (0.04)</b>
<b>Shear:</b>	<b>V.max*=7.5kN &lt; <math>\phi</math>Vvm = 207.4kN (Web area full depth)</b>	<b>OK (0.04)</b>
<b>Deflection:</b>	<b><math>\delta</math>DL = L/164911 (0mm), <math>\delta\psi_s</math>.LL = L/28296 (0mm), <math>\delta</math>Total = L/24152 (0mm), 1kN mid. <math>\delta</math> = 0.0mm</b>	<b>OK</b>
<b>Precamber:</b>	<b>Not required</b>	
<b>Reactions:</b>	<b>(Each end) Rdl = 0.6kN, Rll = 4.5kN, R* = 7.5kN</b>	

**Geometry**

Span (L) =	1200 mm	Eff. Len. (Le) =	600 mm
Centres (cts) =	2000 mm	$\alpha_m$ .t =	1.30
Design at =	M mm from LHS, (M)ax, (S)eg		
Span type =	S (S)ingle, (D)ouble		
Effective length (Le) =	600 mm		
$\alpha_m$ =	1.30		

**Loadings**

Floor area =	2.4 m <sup>2</sup>	Live load type =	N (N)ormal, (S)torage, (M)annual
Apply reduction =	N (Y)es, (N)o		AS/NZS 1170.0 - Table 4.1
Floor reduction ( $\psi_a$ ) =	1.00 AS/NZS 1170.1 - Cl 3.4.2		
<b>Uniform dead loads</b>			
Floor dead load (wdl) =	0.40 kPa *	2000 mm +	kN/m = 0.80 kN/m
Wall dead load (wdl) =	kPa *	2000 mm +	kN/m = 0.00 kN/m
Other dead load (wdl) =	kPa *	mm +	kN/m = 0.00 kN/m
Include S.Wt =	Y (Y)es, (N)o		S.Wt = 0.23 kN/m
			$\Sigma$ wdl = 1.03 kN/m
<b>Uniform live loads</b>			
Floor live load (wll) =	3.00 kPa * $\psi_a$ *	2000 mm +	kN/m = 6.00 kN/m
Other live load (wll) =	kPa *	2000 mm +	kN/m = 0.00 kN/m
Alternate point live load =	4.50 kN (critical)	Distr. to	1 members $\Sigma$ wll = 7.50 kN/m
<b>Point loads</b>			
Dead load (pdl) =	kN	Position =	600 mm from LHS
Live load (pll) =	kN	Shear using PL at support =	N (Y)es, (N)o
Short term LL factor ( $\psi_{su}$ ) =	0.70	( $\psi_{sp}$ ) =	1.00
$w^* = 1.2 * wdl + 1.5 * wll =$	12.49 kN/m	$M^* =$	2.2 kNm (Max at 600mm)
$p^* = 1.2 * pdl + 1.5 * pll =$	0.00 kN	$V^* =$	7.5 kN

**Capacity**

Description =	200x75PFC (G300)	Warping constant (Iw) =	10.5 x10 <sup>9</sup> mm <sup>6</sup>
Flange yield (fyf) =	300 MPa	Torsional constant (J) =	105 x10 <sup>3</sup> mm <sup>4</sup>
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	221 x10 <sup>3</sup> mm <sup>3</sup>
Area (Ag) =	2920 mm <sup>2</sup>	Effective section mod. (Zey) =	46.7 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Ix) =	19.1 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4
Stiffness (Iy) =	1.65 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4
$\phi$ =	0.9 Table 3.4		
$M_{sx} = \min(fyf, fyw) * Z_{ex} =$	66.3 kNm - Cl 5.2.1	$\phi M_{sx} =$	59.7 kNm
Positive: $M_{oa.p} =$	772.6 kNm $\alpha_s.t = 0.989$	$\alpha_m.t = 1.30$	$\phi M_{sy} =$
Negative: $M_{oa.n} =$	N.A.		12.6 kNm - Cl 5.2.1
			$\phi M_{bx.p} =$
			59.7 kNm (= $\phi M_{sx}$ )

**Deflections**

Ireq'd DL (L/360) =	0.0 x10 <sup>6</sup> mm <sup>4</sup>	$\delta$ DL =	0.0 mm	Span / 164911
Ireq'd $\psi_s$ .LL (L/360) =	0.2 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\psi_s$ . $\delta$ LL =	0.0 mm	Span / 28296
Ireq'd (DL+ $\psi_s$ .LL) (L/250) =	0.2 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\delta$ Total =	0.0 mm	Span / 24152
Max. precamber (0.3%*span) =	4 mm	Min. precamber =	15 mm	
Precamber 80% of $\delta$ DL =	0 mm	Adopted precamber =	0 mm	
		1kN midspan $\delta$ =	0.0 mm	

**STEEL FLOOR BEAM V5.07**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Floor Beam FB4) 250x90PFC (G300)</b>	
<b>Bending:</b>	<b>M*(max) = 61.4kNm &lt; <math>\phi</math>Mb(2650, <math>\alpha_m=1.30</math>) = 102.4kNm</b>	<b>OK (0.60)</b>
	<b>(Single span)</b>	
<b>Shear:</b>	<b>V.max*=36.0kN &lt; <math>\phi</math>Vvm = 345.6kN (Web area full depth)</b>	<b>OK (0.10)</b>
<b>Deflection:</b>	<b><math>\delta</math>DL = L/2965 (2mm), <math>\delta\psi</math>s.LL = L/582 (9mm), <math>\delta</math>Total = L/486 (11mm), 1kN mid. <math>\delta</math> = 0.3mm</b>	<b>OK</b>
<b>Precamber:</b>	<b>Not required</b>	
<b>Reactions:</b>	<b>(1 End) Rdl.max = 3.9kN, Rll.max = 20.9kN, R.max* = 36.0kN</b>	
	<b>(1 End) Rdl.min = 2.5kN, Rll.min = 13.2kN, R.min* = 22.9kN</b>	

**Geometry**

Span (L) =	5300 mm	Eff. Len. (Le) =	2650 mm
Centres (cts) =	mm	$\alpha_m.t$ =	1.30
Design at =	M mm from LHS, (M)ax, (S)eg		
Span type =	S (S)ingle, (D)ouble		
Effective length (Le) =	2650 mm		
$\alpha_m$ =	1.30		

**Loadings**

Floor area =	0.0 m <sup>2</sup>	Live load type =	N (N)ormal, (S)torage, (M)annual
Apply reduction =	N (Y)es, (N)o		AS/NZS 1170.0 - Table 4.1
Floor reduction ( $\psi_a$ ) =	1.00 AS/NZS 1170.1 - Cl 3.4.2		

**Uniform dead loads**

Floor dead load (wdl) =	0.40 kPa *	0 mm +	kN/m =	0.00 kN/m
Wall dead load (wdl) =	kPa *	0 mm +	kN/m =	0.00 kN/m
Other dead load (wdl) =	kPa *	mm +	kN/m =	0.00 kN/m
Include S.Wt =	Y (Y)es, (N)o		S.Wt =	0.36 kN/m
			$\Sigma$ wdl =	0.36 kN/m

**Uniform live loads**

Floor live load (wll) =	3.00 kPa * $\psi_a$ *	0 mm +	kN/m =	0.00 kN/m
Other live load (wll) =	kPa *	0 mm +	kN/m =	0.00 kN/m
Alternate point live load =	4.50 kN (critical)	Distr. to	1 members	$\Sigma$ wll = 1.70 kN/m

**Point loads**

Dead load (pdl) =	4.6 kN	DL	LL	Position =	3455 mm from LHS
Live load (pll) =	25.1 kN	$\Sigma$ 4.6	25.1	using PL at support =	N (Y)es, (N)o

Short term LL factor ( $\psi_{su}$ ) =	0.70	( $\psi_{sp}$ ) =	1.00
$w^* = 1.2*w_{dl} + 1.5*w_{ll} =$	2.97 kN/m	$M^* =$	61.4 kNm (Max at 3455mm)
$p^* = 1.2*p_{dl} + 1.5*p_{pl} =$	43.18 kN	$V^* =$	36.0 kN

**Capacity**

Description =	250x90PFC (G300)	Warping constant (Iw) =	35.9 x10 <sup>9</sup> mm <sup>6</sup>	
Flange yield (fyf) =	300 MPa	Torsional constant (J) =	248 x10 <sup>3</sup> mm <sup>4</sup>	
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	421 x10 <sup>3</sup> mm <sup>3</sup>	
Area (Ag) =	4520 mm <sup>2</sup>	Effective section mod. (Zey) =	88.7 x10 <sup>3</sup> mm <sup>3</sup>	
Stiffness (Ix) =	45.1 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4	
Stiffness (Iy) =	3.64 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4	
$\phi$ =	0.9 Table 3.4			
$M_{sx} = \min(fyf, fyw) * Z_{ex} =$	126.3 kNm - Cl 5.2.1	$\phi M_{sx} =$	113.7 kNm	
Positive: Moa.p =	175.0 kNm $\alpha_s.t = 0.693$	$\alpha_m.t = 1.30$	$\phi M_{sy} =$	23.9 kNm - Cl 5.2.1
Negative: Moa.n =	N.A.		$\phi M_{bx.p} =$	102.4 kNm

**Deflections** ( $\delta \pm 2.5\%$  with off-centre PL)

Ireq'd DL (L/360) =	5.5 x10 <sup>6</sup> mm <sup>4</sup>	$\delta$ DL =	1.8 mm	Span / 2965
Ireq'd $\psi$ s.LL (L/360) =	27.9 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\psi$ s. $\delta$ LL =	9.1 mm	Span / 582
Ireq'd (DL+ $\psi$ s.LL) (L/250) =	23.2 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\delta$ Total =	10.9 mm	Span / 486
Max. precamber (0.3%*span) =	16 mm	Min. precamber =	15 mm	
Precamber 80% of $\delta$ DL =	1 mm	Adopted precamber =	0 mm	
1kN midspan $\delta$ =			0.3 mm	

**ANALYSIS V5.05**

Deery Consulting Structural Engineer

**Geometry for (FB4): Steel Simple Beam**

Description = 250x90PFC (G300) Ix = 45.1 x10<sup>6</sup> mm<sup>4</sup>  
 Span (L) = 5300 mm Axis = X (X),(Y)  
 Span type = S (S)imple,(E)xt,(I)nt,(C)ant,(P)rop,(F)ixed,(O)ther Ag = 4520 mm<sup>2</sup>  
 Material type = S (T)imber,(S)teel,(C)onc.,(SC)comp. steel,(O)ther Density = 78.6 kN/m<sup>3</sup>  
E = 200000 MPa

**Loading**

	DL	LL	
FB1	4.6	25.1	: L L
Σ	4.6	25.1	

Uniform loads	Uniform loads (kN/m)		
	UDL	Partial 1	Partial 2
Dead load (wdl) =			
Live load (wll) =			
Start from LHS (mm) =	0		
End from LHS (mm) =	5300		
S.Wt =	0.36	kN/m	
Ultimate load (w*) =	0.43	0.00	0.00

Point loads	Point loads (kN)		
	PL 1	PL 2	PL 3
Dead load (pdl) =	4.59	3.95	
Live load (pll) =	25.11	2.25	
Pos. from LHS (mm) =	3450	1000	
Ultimate load (p*) =	43.18	8.12	0.00
Include S.Wt =	Y (Y)es,(N)o		
Strength loadcase =	C (D)ead Only,(C)omb.		

Live Load type = Floor (Steel)  
 Short term LL (Ψsu) = 0.70 (Ψsp) = 1.00  
 Long term LL (Ψlu) = 0.40 (Ψlp) = 0.60  
 Actual LL (Ψsa) = 1.00 (Ψla) = 0.60

**Results at midspan**

Position of result (x) = 2650 mm

1.20\*G+1.50\*Q analysed

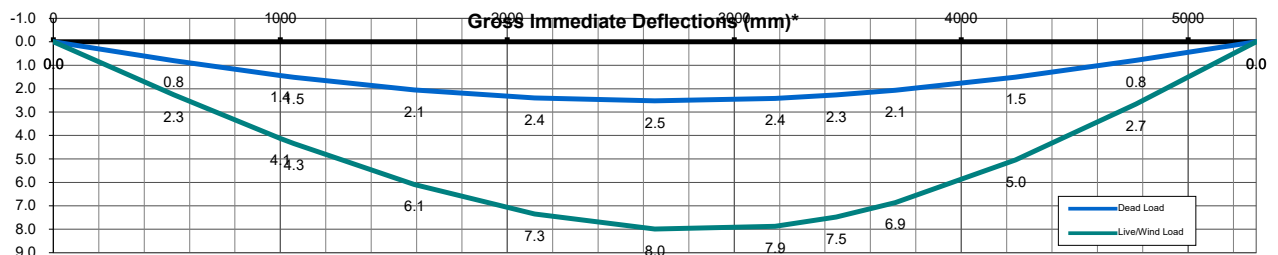
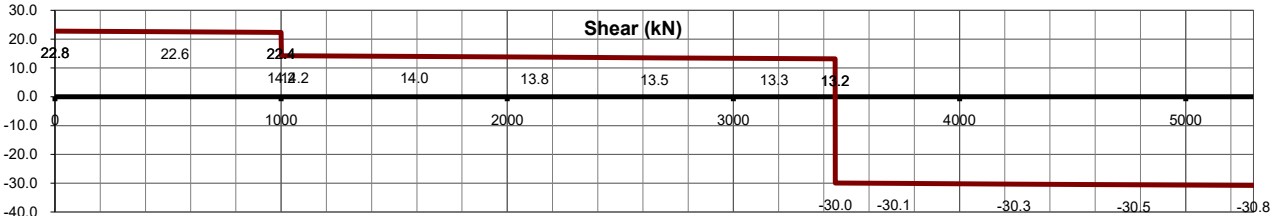
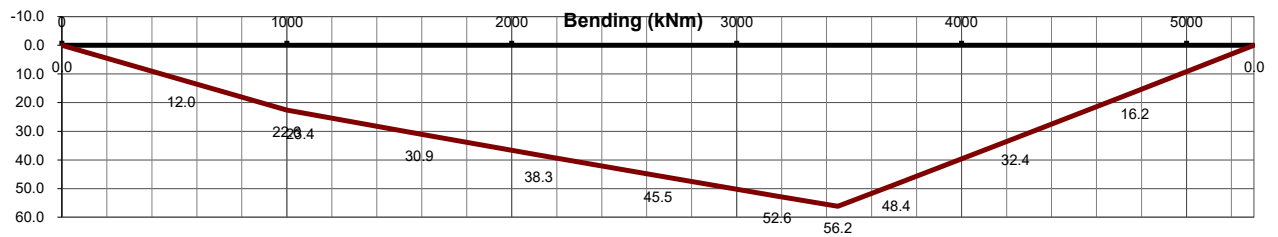
	Left	At x	Right	Max	At	Min	At	Units
Rdl	5.75		4.68					kN
Rll	10.59		16.77					kN
R*	22.79		30.77					kN
M*	0.00	45.50	0.00	56.19	3450	0.00	0	kNm
V*	22.79	13.54	-30.77	30.77	5300			kN
δdl	0.00	2.52	0.00	2.52	2650	0.00	0	mm
δll	0.00	7.99	0.00	7.99	2650	0.00	0	mm
δdl+Ψs*δll	0.00	10.51	0.00	10.51	2650	0.00	0	mm

Span / 2101  
663  
504

δPlI/δTot.II = 1.000

**Graphs**

\* Deflections are Gross Ig immediate - assessment of long term effects to be considered



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**STEEL MEMBER V5.15**

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<b>Section:</b>	<b>(FB4) 250x90PFC (G300)</b>	
<b>Bending:</b>	<b><math>M_x^* = 56.2 \text{ kNm} &lt; \phi M_b(5300, \alpha_m=1.30) = 65.3 \text{ kNm}</math>, <math>\phi M_b(\alpha_m=1) = 50.2 \text{ kNm}</math></b>	<b>OK (0.86)</b>
	<b>No minor bending</b>	
<b>Shear:</b>	<b><math>V_x^* = 30.8 \text{ kN} &lt; \phi V_{vm} = 345.6 \text{ kN}</math> (Web area full depth)</b>	<b>OK (0.09)</b>
	<b>No minor shear</b>	
<b>Compression:</b>	<b>No compression</b>	
<b>Tension:</b>	<b>No tension</b>	
<b>Torsion:</b>	<b>No torsion</b>	
<b>Deflection:</b>	<b><math>\delta DL = L/2101</math> (3mm), <math>\delta \Psi_{s.LL} = L/663</math> (8mm), <math>\delta Total = L/504</math> (11mm) at 2650mm from LHS</b>	<b>OK</b>

**Bending & Shear at critical locations - Section 5** Max. restraint (2.5% flange force) = 6.0 kN

(M\* to include first order amplification as required - Cl 4.4.2)

Analysis values = C (Manual), (L)eft, (P)osition (X) from analysis, (R)ight, (C)ritical

Refer to the analysis output

Analysis Axis = X (X),(Y)

Major bending ( $M_x^*$ ) =	56.2 kNm	Minor bending ( $M_y^*$ ) =	0 kNm
Minor bending ( $M_y^*$ ) =	0.000 kNm	Torsion ( $M_z^*$ ) =	0 kNm
Shear Force ( $V_x^*$ ) =	30.8 kN	Shear Force ( $V_y^*$ ) =	0 kN
Shear Force ( $V_y^*$ ) =	0.000 kN	Span / Segment Length (L) =	5300 mm
Effective length factor ( $k_e$ ) =	Calc	Moment modification factor ( $\alpha_m$ ) =	1.30
$k_e = (k_t=1.00) * (k_l=1.00) * (k_r=1.00) =$	1.00 (From Le Tab)	Bending (x) =	OK (0.86)
Effective length ( $L_e = L * k_e$ ) =	5300 mm	Shear =	OK (0.09)
$\phi =$	0.9 Table 3.4	$\phi M_{syR} =$	24.0 kNm
$\phi M_{sx} =$	113.7 kNm	$I_x =$	45.1 x10 <sup>6</sup> mm <sup>4</sup>
$\phi M_{bx}(\alpha_m=1) =$	50.2 kNm	S.Wt =	0.355 kN/m
$\phi M_{bx} =$	65.3 kNm		
$\phi M_{syL} =$	23.9 kNm		
$\phi V_v =$	345.6 kN		
$\phi V_{vm} =$	345.6 kN		
$\phi M_z =$	2.68 kNm		

**Compression - Section 6 (No compression)**

Axial compression ( $N_c^*$ ) =	0.0 kN	Axial compression ( $N_c^*$ ) =	0 kN
Major axis length ( $L_x$ ) =	5300 mm	Eff. X length factor ( $k_{ex}$ ) =	1.00
Minor axis length ( $L_y$ ) =	2700 mm	Eff. Y length factor ( $k_{ey}$ ) =	1.00
Braced or Sway member =	S (B)raccd, (S)way	Major axis effective length ( $L_{ex}$ ) =	5300 mm
$\phi N_s =$	1220.4 kN	Minor axis effective length ( $L_{ey}$ ) =	2700 mm
$\phi N_{cx}(k_{ex}=1.00) =$	925.2 kN		
In-Plane $\phi N_{cx}(k_{ex}=1.00) =$	925.2 kN		
$\phi N_{cy}(k_{ey}=1.00) =$	563.3 kN		
In-Plane $\phi N_{cy}(k_{ey}=1.00) =$	563.3 kN		
$\phi N_c =$	563.3 kN		

**Tension - Section 7 (No tension)**

Axial tension ( $N_t^*$ ) =	0.0 kN	Axial tension ( $N_t^*$ ) =	0 kN
$k_t =$	1.00 Table 7.3.2		
$\phi N_t =$	1220.4 kN		

**Combined**

$\phi M_{rxt} =$	113.7 kNm	$\phi M_{ixt} =$	113.7 kNm	$\phi M_{oxt} =$	65.3 kNm
$\phi M_{rxc} =$	113.7 kNm	$\phi M_{ixc} =$	113.7 kNm	$\phi M_{oxc} =$	65.3 kNm
$\phi M_{ryt} =$	23.9 kNm	$\phi M_{iyc} =$	23.9 kNm	$\phi M_{tx} =$	65.3 kNm
$\phi M_{ryc} =$	23.9 kNm			$\phi M_{cx} =$	65.3 kNm

**STEEL FLOOR BEAM V5.07**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Floor Beam FB5) 200x75PFC (G300)</b>	
<b>Bending:</b>	<b>M*(max) = 34.0kNm &lt; <math>\phi</math>Mb(1350, <math>\alpha_m=1.30</math>) = 59.7kNm</b>	<b>OK (0.57)</b>
	(Single span)	
<b>Shear:</b>	<b>V.max*=28.7kN &lt; <math>\phi</math>Vvm = 207.4kN (Web area full depth)</b>	<b>OK (0.14)</b>
<b>Deflection:</b>	<b><math>\delta</math>DL = L/5050 (1mm), <math>\delta\psi_s.LL = L/849</math> (3mm), <math>\delta</math>Total = L/727 (4mm), 1kN mid. <math>\delta = 0.1</math>mm</b>	<b>OK</b>
<b>Precamber:</b>	<b>Not required</b>	
<b>Reactions:</b>	<b>(Each end) Rdl = 2.6kN, Rll = 17.1kN, R* = 28.7kN</b>	

**Geometry**

Span (L) =	2700 mm	Eff. Len. (Le) =	1350 mm
Centres (cts) =	mm	$\alpha_m.t =$	1.30
Design at =	M mm from LHS, (M)ax, (S)eg		
Span type =	S (S)ingle, (D)ouble		
Effective length (Le) =	1350 mm		
$\alpha_m =$	1.30		

**Loadings**

Floor area =	0.0 m <sup>2</sup>	Live load type =	N (N)ormal, (S)torage, (M)annual
Apply reduction =	N (Y)es, (N)o		AS/NZS 1170.0 - Table 4.1
Floor reduction ( $\psi_a$ ) =	1.00 AS/NZS 1170.1 - Cl 3.4.2		

**Uniform dead loads**

Floor dead load (wdl) =	0.40 kPa *	0 mm +	kN/m =	0.00 kN/m
Wall dead load (wdl) =	kPa *	0 mm +	kN/m =	0.00 kN/m
Other dead load (wdl) =	kPa *	mm +	kN/m =	0.00 kN/m
Include S.Wt =	Y (Y)es, (N)o		S.Wt =	0.23 kN/m
			$\Sigma wdl =$	0.23 kN/m

**Uniform live loads**

Floor live load (wll) =	3.00 kPa * $\psi_a$ *	0 mm +	kN/m =	0.00 kN/m
Other live load (wll) =	kPa *	0 mm +	kN/m =	0.00 kN/m
Alternate point live load =	4.50 kN (critical)	Distr. to	1 members	$\Sigma wll =$ 3.33 kN/m

**Point loads**

Dead load (pdl) =	4.6 kN	DL	LL	Position =	1350 mm from LHS	
Live load (pll) =	25.1 kN	FB1	4.6	25.1 : L   L		
		$\Sigma$	4.6	25.1	using PL at support =	N (Y)es, (N)o

Short term LL factor ( $\psi_{su}$ ) =	0.70	( $\psi_{sp}$ ) =	1.00
$w^* = 1.2 * wdl + 1.5 * wll =$	5.28 kN/m	$M^* =$	34.0 kNm (Max at 1350mm)
$p^* = 1.2 * pdl + 1.5 * pll =$	43.18 kN	$V^* =$	28.7 kN

**Capacity**

Description =	200x75PFC (G300)	Warping constant (Iw) =	10.5 x10 <sup>9</sup> mm <sup>6</sup>
Flange yield (fyf) =	300 MPa	Torsional constant (J) =	105 x10 <sup>3</sup> mm <sup>4</sup>
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	221 x10 <sup>3</sup> mm <sup>3</sup>
Area (Ag) =	2920 mm <sup>2</sup>	Effective section mod. (Zey) =	46.7 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Ix) =	19.1 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4
Stiffness (Iy) =	1.65 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4
$\phi =$	0.9 Table 3.4		
$M_{sx} = \min(fyf, fyw) * Z_{ex} =$	66.3 kNm - Cl 5.2.1	$\phi M_{sx} =$	59.7 kNm
Positive: Moa.p =	188.0 kNm $\alpha_s.t = 0.849$	$\alpha_m.t =$	1.30
Negative: Moa.n =	N.A.	$\phi M_{sy} =$	12.6 kNm - Cl 5.2.1
		$\phi M_{bx.p} =$	59.7 kNm (= $\phi M_{sx}$ )

**Deflections**

Ireq'd DL (L/360) =	1.4 x10 <sup>6</sup> mm <sup>4</sup>	$\delta$ DL =	0.5 mm	Span / 5050
Ireq'd $\psi_s.LL$ (L/360) =	8.1 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\psi_s.\delta LL =$	3.2 mm	Span / 849
Ireq'd (DL+ $\psi_s.LL$ ) (L/250) =	6.6 x10 <sup>6</sup> mm <sup>4</sup> < Critical	$\delta$ Total =	3.7 mm	Span / 727
Max. precamber (0.3%*span) =	8 mm	Min. precamber =	15 mm	
Precamber 80% of $\delta$ DL =	0 mm	Adopted precamber =	0 mm	
		1kN midspan $\delta =$	0.1 mm	

**TIMBER FLOOR BEAM V5.10**

Deery Consulting Structural Engineer

<b>Beam:</b>	<b>(Floor Joist FJ1 (timber)) 300mm x 63mm Smart LVL13 (Varies) (Single span)</b>	
<b>Bending:</b>	<b>M(dI)* = 4.83kNm (+ve) &lt; ϕM(dI) = 12.64kNm, M* = 10.25kNm (+ve) &lt; ϕM = 19.58kNm</b>	<b>OK (0.38,0.52)</b>
<b>Shear:</b>	<b>V(dI)* = 3.61kN &lt; ϕV(dI) = 34.11kN, V* = 7.66kN &lt; ϕV = 47.88kN</b>	<b>OK (0.11,0.16)</b>
<b>Deflection:</b>	<b>δDL = L/644 (8mm), δΨs.LL = L/697 (8mm), δTotal = L/335 (16mm), 1kN mid. δ = 1.7mm</b>	<b>Warning (vibr.)</b>
<b>Reactions:</b>	<b>(Each end) Rdl = 0.76kN, Rll = 4.50kN, R* = 7.66kN</b>	

**Geometry** (House member affecting ≤ 25m<sup>2</sup> or secondary building member)

Category =	1 (1) House, (2) Primary building elements, (3) Important	
House =	Y (Y)es,(N)o	Edge restrained = C (T)ension, (C)ompression, (B)oth
Span (L) =	5350 mm	Lay.t (Top) = 2675 mm
Centres (cts) =	450 mm	
Span type =	S (S)ingle,(D)ouble	

**Loadings**

Floor area (A) =	2.41 m <sup>2</sup>	Live load type =	N (N)ormal, (S)torage, (M)annual
			AS/NZS 1170.0 - Table 4.1

**Uniform dead loads**

Floor dead load (wdl) =	0.40 kPa *	450 mm +	kN/m =	0.18 kN/m
Wall dead load (wdl) =	kPa *	450 mm +	kN/m =	0.00 kN/m
Other dead load (wdl) =	kPa *	mm +	kN/m =	0.00 kN/m
Include S.Wt =	Y (Y)es,(N)o		S.Wt =	0.10 kN/m
			Σwdl =	0.28 kN/m

**Uniform live loads**

Floor live load (wll) =	3.00 kPa *	450 mm +	kN/m =	1.35 kN/m
Other live load (wll) =	kPa *	450 mm +	kN/m =	0.00 kN/m
Alternate point live load =	4.50 kN (critical)	Distr. to	1 members	Σwll = 1.68 kN/m

**Point loads**

Dead load (pdl) =	kN	Position =	2675 mm from LHS
Live load (pll) =	kN	Shear using PL at support =	N (Y)es,(N)o

Short term LL factor (Ψsu) =	1.00	(Ψsp) =	1.00
Long term LL factor (Ψlu) =	0.33 / 0.40 (wdl*)	(Ψlp) =	0.33 / 0.40 (pdl*)
wdl* = 1.2*wdl+1.5*Ψlu*wll =	1.35 kN/m	M(dI+Ψl.II)* =	4.83 kNm (Max at 2675mm)
w* = 1.2*wdl+1.5*wll =	2.86 kN/m	M* =	10.25 kNm (Max at 2675mm)
pdl* = 1.2*pdl+1.5*Ψlp*pll =	0.00 kN	V(dI+Ψl.II)* =	3.61 kN
p* = 1.2*pdl+1.5*pll =	0.00 kN	V* =	7.66 kN

**Bending and Shear capacity - Cl 3.2.1 & Cl 3.2.5**

Member =	300mm x 63mm Smart LVL13	Area (A) =	18900 mm <sup>2</sup>
Description =	LVL13 seasoned softwood	Section modulus (Zx) =	945 x10 <sup>3</sup> mm <sup>3</sup>
Design depth (dD) =	300 mm	Stiffness (Ix) =	141.8 x10 <sup>6</sup> mm <sup>4</sup>
Design width (dW) =	63 mm	Modulus of elasticity (E) =	13200 MPa - Cl 8.3

S1 = 1.25*dD/dW*(Lay.t/dD) <sup>0.5</sup> =	17.77 For top comp. edge restrained - Cl 3.2.3.2(a)		
k12d = 1.5-0.05*pbd*S1 =	0.626 for 10 ≤ pbd*S1 ≤ 20 - Cl 3.2.4	f'b =	39.4 MPa
k12 = 1.5-0.05*pb*S1 =	0.691 for 10 ≤ pb*S1 ≤ 20 - Cl 3.2.4	f's =	5.0 MPa
Strength reduction factor (ϕ) =	0.95 Table 2.1	Material constant (pbd) =	0.98 (rbd=0.25)
ϕM(dI) = ϕ*(k1=0.57)*k4*k6*k9*k12d*f'b*Zx =	12.64 kNm	Material constant (pb) =	0.91 (rb=0.88)
ϕM = ϕ*k1*k4*k6*k9*k12*f'b*Zx =	19.58 kNm	Duration factor (k1) =	0.80
ϕV(dI) = ϕ*(k1=0.57)*k4*k6*f's*(2/3*A) =	34.11 kN	Moisture factor (k4) =	1.00
ϕV = ϕ*k1*k4*k6*f's*(2/3*A) =	47.88 kN	Temp. factor (k6) =	1.00
		Sharing factor (k9) =	1.00
		Size modifier (mod.b) =	1.00
		Size modifier (mod.s) =	1.00

**Deflections**

Req'd j2.(DL+Ψl.LL) (16mm) =	73.5 x10 <sup>6</sup> mm <sup>4</sup>	Duration factor (j2) =	2.0
Req'd Ψs.LL (12mm) =	90.6 x10 <sup>6</sup> mm <sup>4</sup>	j2.(DL+Ψl.LL) =	8.3 mm
Req'd Total (L/250) =	105.8 x10 <sup>6</sup> mm <sup>4</sup>	< Critical Ψs.δLL =	7.7 mm
		δTotal =	16.0 mm
		1kN midspan δ =	1.7 mm

**STEEL MULLION V5.04**

Deery Consulting Structural Engineer

<b>Member:</b>	<b>(Door mullion DM1) 180x75PFC (G300) - No flybracing</b>	
<b>Bending:</b>	<b>M.in* = 14.7kNm &lt; <math>\phi Mb(1400, \alpha m=1.00) = 41.4kNm</math></b>	<b>OK (0.35)</b>
	<b>M.out* = 13.9kNm &lt; <math>\phi Mb(5600, \alpha m=1.13) = 20.9kNm</math></b>	<b>OK (0.66)</b>
<b>Combined:</b>	<b>In-plane = 0.30, Out-of-plane = 0.66</b>	<b>OK (0.66)</b>
<b>Deflection:</b>	<b><math>\delta W L s . i n = L / 484</math> (12mm), <math>\delta W L s . o u t = L / 513</math> (11mm)</b>	<b>OK</b>
<b>Reactions:</b>	<b>(Each end) R.in* = 10.5kN, (Each end) R.out* = 9.9kN</b>	

**Geometry**

Length (L=Lex) =	5600 mm	Use for Ley =	G (G)irts,(F)lybraces,(C)ustom
Centres (cts) =	3200 mm		
Girt centres = eff. length (Le) =	1400 mm (Outside flange)	Effective length (Ley) =	1400 mm
Inward ( $\alpha m$ ) =	1.00	Eff. length inside (Leo) =	5600 mm
Inward ( $\alpha m$ ) =	1.00	No. Flybraces to inside flange =	> 500 for seg length)
		Outward ( $\alpha m . o$ ) =	1.13

**Loadings** Wind area reduction not applied

Wall area =	17.9 m <sup>2</sup>	Apply wind reduction =	N (N)one,(S)ide,(W)ind,(L)ee
Ws/Wu =	0.68	Area reduction (ka) =	1.00 AS 1170.2 Table 5.4
Ult. wind load in (Wu.in) =	1.30 kPa		
cpe =	0.7	cpi =	0.2
		w.in* =	3.74 kN/m
Ult. wind load out (Wu.out) =	1.30 kPa		
cpe =	0.65	cpi =	0.2
		w.out* =	3.54 kN/m
Axial compression (Nc*) =	0.0 kN	Eccentricity (ecc) =	90 mm (+ve increases out)
		D/2 =	90 mm
<b>Horz. Point loads</b>			
Wind load (pwl.in*) =	kN	Position =	2800 mm from bottom
Wind load (pwl.out*) =	kN		
w.in* =	3.74 kN/m	w.out* =	3.54 kN/m
p.in* =	0.00 kN	p.out* =	0.00 kN
M.in* =	14.7 kNm	M.out* =	13.9 kNm
M.(in+e)* =	14.7 kNm	M.(out+e)* =	13.9 kNm
		R.in* =	10.5 kN
		R.out* =	9.9 kN
		M.e* =	0.00 kNm
		M.(crit+e)* =	14.7 kNm

**Capacity**

Description =	180x75PFC (G300)		Warping constant (Iw) =	7.75 x10 <sup>9</sup> mm <sup>6</sup>	
Flange yield (fyf) =	300 MPa		Torsional constant (J) =	84.5 x10 <sup>3</sup> mm <sup>4</sup>	
Web yield (fyw) =	320 MPa		Effective section mod. (Zex) =	182 x10 <sup>3</sup> mm <sup>3</sup>	
Area (Ag=An) =	2660 mm <sup>2</sup>		Effective section mod. (Zey) =	44.9 x10 <sup>3</sup> mm <sup>3</sup>	
Stiffness (Ix) =	14.1 x10 <sup>6</sup> mm <sup>4</sup>		Elastic modulus (E) =	200000 MPa - Cl 1.4	
Stiffness (Iy) =	1.51 x10 <sup>6</sup> mm <sup>4</sup>		Shear modulus (G) =	80000 MPa - Cl 1.4	
ab =	0.5 (comp)		rx =	72.8 mm	
$\phi$ =	0.9 Table 3.4		ry =	23.8 mm	
<b>Bending</b>			$\phi M s y = \phi * \min (f y f , f y w ) * Z e y =$	12.1 kNm - Cl 5.2.1	
Msx = min(fyf,fyw)*Zex =	54.6 kNm - Cl 5.2.1		$\phi M s x =$	49.1 kNm	
Inward: Moa =	148.8 kNm	$\alpha s = 0.842$	$\alpha m = 1.00$	$\phi M b x =$	41.4 kNm
Outward: Moa =	26.2 kNm	$\alpha s = 0.376$	$\alpha m . o = 1.13$	$\phi M b x . o =$	20.9 kNm < Critical
<b>Compression</b>			$\phi N s = \phi * k f * A n * f y =$	718.2 kN	
$\alpha c x =$	0.584	$\phi N c x = \phi N c x 1 = \alpha c x * \phi N s =$	419.4 kN		
$\alpha c y =$	0.718	$\phi N c y = \phi N c y 1 = \alpha c y * \phi N s =$	515.5 kN		
Ley =	1400 mm	$\phi N c = \min (\phi N c x , \phi N c y ) =$	419.4 kN		
<b>Combined</b>			$\phi M r x c = \phi M s x * ( 1 - N c * / \phi N s ) =$	49.1 kNm	
		$\phi M i x c = \phi M s x * ( 1 - N c * / \phi N c x 1 ) =$	49.1 kNm	OK (0.30)	
		$\phi M o x c = \phi M b x . o * ( 1 - N c * / \phi N c y ) =$	20.9 kNm	OK (0.66)	
		In-plane member ratio = $M x * / \phi M s x + N c * / \phi N c x 1 =$	0.30	OK (0.30)	
		Out-of-plane member ratio = $M x * / \phi M b x . o + N c * / \phi N c y =$	0.66	OK (0.66)	

**Deflections**

Ireq'd WLs.in (L/250) =	7.3 x10 <sup>6</sup> mm <sup>4</sup>	< Critical	$\delta W L s . i n =$	11.6 mm	Span / 484
Ireq'd WLs.out (L/250) =	6.9 x10 <sup>6</sup> mm <sup>4</sup>		$\delta W L s . o u t =$	10.9 mm	Span / 513

**STEEL MULLION V5.04**

Deery Consulting Structural Engineer

<b>Member:</b>	(Door mullion DM1_02) 180x75PFC (G300) - No flybracing	
<b>Bending:</b>	M.in* = 14.8kNm < $\phi Mb(1400, \alpha m=1.00) = 41.4$ kNm	<b>OK (0.36)</b>
	M.out* = 13.9kNm < $\phi Mb(5850, \alpha m=1.13) = 20.1$ kNm	<b>OK (0.69)</b>
<b>Combined:</b>	In-plane = 0.30, Out-of-plane = 0.69	<b>OK (0.69)</b>
<b>Deflection:</b>	$\delta W L s . i n = L / 461$ (13mm), $\delta W L s . o u t = L / 488$ (12mm)	<b>OK</b>
<b>Reactions:</b>	(Each end) R.in* = 10.1kN, (Each end) R.out* = 9.5kN	

**Geometry**

Length (L=Lex) =	5850 mm	Use for Ley =	G (G)irts,(F)lybraces,(C)ustom
Centres (cts) =	2950 mm		
Girt centres = eff. length (Le) =	1400 mm (Outside flange)	Effective length (Ley) =	1400 mm
Inward ( $\alpha m$ ) =	1.00	Eff. length inside (Leo) =	5850 mm
Inward ( $\alpha m$ ) =	1.00	No. Flybraces to inside flange =	> 500 for seg length)
		Outward ( $\alpha m . o$ ) =	1.13

**Loadings** Wind area reduction not applied

Wall area =	17.3 m <sup>2</sup>	Apply wind reduction =	N (N)one,(S)ide,(W)ind,(L)ee
Ws/Wu =	0.68	Area reduction (ka) =	1.00 AS 1170.2 Table 5.4
Ult. wind load in (Wu.in) =	1.30 kPa		
cpe =	0.7	cpi =	0.2
		w.in* =	3.45 kN/m
Ult. wind load out (Wu.out) =	1.30 kPa		
cpe =	0.65	cpi =	0.2
		w.out* =	3.26 kN/m
Axial compression (Nc*) =	0.0 kN	Eccentricity (ecc) =	90 mm (+ve increases out)
		D/2 =	90 mm
<b>Horz. Point loads</b>			
Wind load (pwl.in*) =	kN	Position =	2925 mm from bottom
Wind load (pwl.out*) =	kN		
w.in* =	3.45 kN/m	w.out* =	3.26 kN/m
p.in* =	0.00 kN	p.out* =	0.00 kN
M.in* =	14.8 kNm	M.out* =	13.9 kNm
M.(in+e)* =	14.8 kNm	M.(out+e)* =	13.9 kNm
		R.in* =	10.1 kN
		R.out* =	9.5 kN
		M.e* =	0.00 kNm
		M.(crit+e)* =	14.8 kNm

**Capacity**

Description =	180x75PFC (G300)	Warping constant (Iw) =	7.75 x10 <sup>9</sup> mm <sup>6</sup>	
Flange yield (fyf) =	300 MPa	Torsional constant (J) =	84.5 x10 <sup>3</sup> mm <sup>4</sup>	
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	182 x10 <sup>3</sup> mm <sup>3</sup>	
Area (Ag=An) =	2660 mm <sup>2</sup>	Effective section mod. (Zey) =	44.9 x10 <sup>3</sup> mm <sup>3</sup>	
Stiffness (Ix) =	14.1 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4	
Stiffness (Iy) =	1.51 x10 <sup>6</sup> mm <sup>4</sup>	Shear modulus (G) =	80000 MPa - Cl 1.4	
ab =	0.5 (comp)	rx =	72.8 mm	
$\phi$ =	0.9 Table 3.4	ry =	23.8 mm	
<b>Bending</b>		$\phi M s y = \phi * \min (f y f , f y w ) * Z e y =$	12.1 kNm - Cl 5.2.1	
Msx = min(fyf,fyw)*Zex =	54.6 kNm - Cl 5.2.1	$\phi M s x =$	49.1 kNm	
Inward: Moa =	148.8 kNm $\alpha s = 0.842$	$\alpha m = 1.00$	$\phi M b x =$	41.4 kNm
Outward: Moa =	25.1 kNm $\alpha s = 0.363$	$\alpha m . o = 1.13$	$\phi M b x . o =$	20.1 kNm < Critical
<b>Compression</b>		$\phi N s = \phi * k f * A n * f y =$	718.2 kN	
$\alpha c x =$	0.559	$\phi N c x = \phi N c x 1 = \alpha c x * \phi N s =$	401.7 kN	
$\alpha c y =$	0.718	$\phi N c y = \phi N c y 1 = \alpha c y * \phi N s =$	515.5 kN	
Ley =	1400 mm	$\phi N c = \min (\phi N c x , \phi N c y ) =$	401.7 kN	
<b>Combined</b>		$\phi M r x c = \phi M s x * ( 1 - N c * / \phi N s ) =$	49.1 kNm <b>OK (0.30)</b>	
		$\phi M i x c = \phi M s x * ( 1 - N c * / \phi N c x 1 ) =$	49.1 kNm <b>OK (0.30)</b>	
		$\phi M o x c = \phi M b x . o * ( 1 - N c * / \phi N c y ) =$	20.1 kNm <b>OK (0.69)</b>	
		In-plane member ratio = $M x * / \phi M s x + N c * / \phi N c x 1 =$	0.30 <b>OK (0.30)</b>	
		Out-of-plane member ratio = $M x * / \phi M b x . o + N c * / \phi N c y =$	0.69 <b>OK (0.69)</b>	

**Deflections**

Ireq'd WLs.in (L/250) =	7.6 x10 <sup>6</sup> mm <sup>4</sup>	< Critical	$\delta W L s . i n =$	12.7 mm	Span / 461
Ireq'd WLs.out (L/250) =	7.2 x10 <sup>6</sup> mm <sup>4</sup>		$\delta W L s . o u t =$	12.0 mm	Span / 488

**STEEL WALL TIE V5.04**

Deery Consulting Structural Engineer

<b>Member:</b>	(Door header DH1_02) 150x75PFC (G300) (Web horizontal)	
<b>Bending:</b>	Mx.in* = 7.6kNm < $\phi$ Mbx.i(3450, $\alpha$ m=1.13) = 22.6kNm	OK (0.34)
	Mx.out* = 7.2kNm < $\phi$ Mbx.o(3450, $\alpha$ m=1.30) = 25.9kNm	OK (0.28)
	My.vert* = 0.5kNm < $\phi$ Msy.d = 11.1kNm (Biaxial = 0.38)	OK (0.38)
<b>Deflection:</b>	$\delta$ in = L/774 (5mm), $\delta$ out = L/820 (5mm), $\delta$ v = L/1751 (2mm)	OK
<b>H. Reactions:</b>	(Each end) R*in = 7.6kN, R*out = 7.2kN	
<b>V. Reactions:</b>	(Each end) Rdl = 0.4kN, Rv* = 0.5kN	

**Geometry**

Span (L) =	4000 mm	Horz. in segment length (Le) =	3450 mm
Wall loadwidth (wcts) =	3250 mm	$\alpha$ m =	1.13
Number of hangers =	(None)	Horz. out segment length (Leo) =	3450 mm
Web in horizontal dir. =	Y (Y)es,(N)o	$\alpha$ mo =	1.30
Vert. weak axis load dir. =	M Load (A), Load (B),(M)in		

**Loadings**

Ratio Ws/Wu =	0.68 (Refer wind analysis)	S.Wt =	0.177 kN/m
<b>Inward</b>			
Ult. inward wind load (Wu) =	1.30 kPa	w.in* =	3.80 kN/m
cpe =	0.7	cpi =	0.2
<b>Outward</b>			
Ult. outward wind load (Wu) =	1.30 kPa	w.out* =	3.59 kN/m
cpe =	0.65	cpi =	0.2
w.in* =	3.80 kN/m	w.out* =	3.59 kN/m
M.in* =	7.61 kNm	M.out* =	7.18 kNm
Mv* =	0.48 kNm (Self weight)	R.in* =	7.6 kN
		R.out* =	7.2 kN

**Capacity**

Description =	150x75PFC (G300)	Warping constant (Iw) =	4.56 x10 <sup>9</sup> mm <sup>6</sup>
Flange yield (fyf) =	320 MPa	Torsional constant (J) =	56.6 x10 <sup>3</sup> mm <sup>4</sup>
Web yield (fyw) =	320 MPa	Effective section mod. (Zex) =	129 x10 <sup>3</sup> mm <sup>3</sup>
Area (Ag) =	2250 mm <sup>2</sup>	Effective section mod. (Zey) =	38.5 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Ix.in) =	8.34 x10 <sup>6</sup> mm <sup>4</sup>	Effective section mod. (ZeyR) =	38.5 x10 <sup>3</sup> mm <sup>3</sup>
Stiffness (Iy.vert) =	1.29 x10 <sup>6</sup> mm <sup>4</sup>	Elastic modulus (E) =	200000 MPa - Cl 1.4
		Shear modulus (G) =	80000 MPa - Cl 1.4
$\phi$ =	0.9 Table 3.4	$\phi$ Msy.d =	11.1 kNm
Msx = min(fyf, fyw)*Zex =	41.3 kNm - Cl 5.2.1	$\phi$ Msx =	37.2 kNm
Inward: Moa =	33.6 kNm $\alpha$ s = 0.537	$\alpha$ m = 1.13	$\phi$ Mbx.i = 22.6 kNm
Outward: Moa =	33.6 kNm $\alpha$ s = 0.537	$\alpha$ m.o = 1.30	$\phi$ Mbx.o = 25.9 kNm
		Biaxial = max(Mx.in*/ $\phi$ Mbx.i & Mx.out*/ $\phi$ Mbx.o) + (My*/ $\phi$ Msy.d) =	0.38

**Deflections**

<b>Horizontal</b>			
Ireq'd WLS.in (L/360) =	3.9 x10 <sup>6</sup> mm <sup>4</sup>	< Critical	$\delta$ WLS.in = 5.2 mm
Ireq'd WLS.out (L/360) =	3.7 x10 <sup>6</sup> mm <sup>4</sup>		$\delta$ WLS.out = 4.9 mm
<b>Vertical</b>			
Ireq'd DL (L/360) =	0.3 x10 <sup>6</sup> mm <sup>4</sup>		$\delta$ Vert = 2.3 mm
			Span / 774
			Span / 820
			Span / 1751

**TIMBER ROOF BEAM V5.10**

Deery Consulting Structural Engineer

<b>Beam:</b>	(Ceiling joist CJ1) 90mm x 45mm MGP10 (Single span)	
<b>Bending:</b>	$M(dl)^* = 0.30\text{kNm (+ve)} < \phi M(dl) = 0.53\text{kNm}$ , $M^* = 0.30\text{kNm (+ve)} < \phi M = 0.87\text{kNm}$ $Mw^* = 0.00\text{kNm (-ve)} < \phi Mw = 0.93\text{kNm}$	OK (0.57,0.35) OK (0.00)
<b>Shear:</b>	$V(dl)^* = 0.45\text{kN} < \phi V(dl) = 3.60\text{kN}$ , $V^* = 0.45\text{kN} < \phi V = 5.94\text{kN}$ $Vw^* = 0.40\text{kN} < \phi Vw = 6.32\text{kN}$	OK (0.12,0.08) OK (0.06)
<b>Deflection:</b>	$\delta DL = L/217$ (12mm) , $\delta \Psi s.LL = L/228571$ (0mm)	Warning
<b>Reactions:</b>	(Each end) $Rdl = 0.33\text{kN}$ , $Rll = 0.00\text{kN}$ , $Rwl^* = 0.00\text{kN}$ , $R.dn^* = 0.45\text{kN}$ , $R.up^* = -0.40\text{kN (Down)}$	

**Geometry** (House member affecting  $\leq 25\text{m}^2$  or secondary building member)

Category =	1 (1) House, (2) Primary building elements, (3) Important	House =	N (Y)es,(N)o
Span (L) =	2700 mm	Edge restrained (down) =	C (T)ension,(C)ompression,(B)oth
Centres (cts) =	900 mm	Lay.t (Top) =	900 mm
Span type =	S (S)ingle,(D)ouble		

**Loadings**

Roof area (A) =	2.43 m <sup>2</sup>	Apply wind reduction =	Y (Y)es,(N)o
LL = 1.8/A+0.12 $\geq$ 0.25 =	0.86 kPa	Roof reduction (Ka) =	1.00 AS/NZS 1170.2, Table 5.4
		Ratio Ws/Wu =	0.68 (Refer wind analysis)

**Uniform dead loads**

Roof dead load (wdl) =	0.25 kPa *	900 mm +	kN/m =	0.23 kN/m
Other dead load (wdl) =	kPa *	900 mm +	kN/m =	0.00 kN/m
Include S.Wt =	Y (Y)es,(N)o		S.Wt =	0.02 kN/m
			$\Sigma wdl =$	0.25 kN/m

**Uniform live loads** No alternative point load specified

Roof live load (wll) =	0.86 kPa *	900 mm +	kN/m =	0.77 kN/m
Other live load (wll) =	-0.86 kPa *	900 mm +	kN/m =	-0.77 kN/m
Alternate point live load =	kN	Distr. to	1 members	$\Sigma wll =$
				0.00 kN/m

**Uniform wind loads**

Ult. wind load (Wu) =	kPa *	900 mm		
Cp,e =	0.9	Cp,i =	0.2	$\Sigma wwll^* =$
				0.00 kN/m

**Point loads**

Dead load (pdl) =	kN	Position =	1350 mm from LHS
Live load (pll) =	kN	Shear using PL at support =	Y (Y)es,(N)o
Wind load (pwl*) =	kN (-ve up)		
$w(dl)^* = 1.35 * wdl =$	0.33 kN/m	$M(dl)^* =$	0.30 kNm (Max at 1350mm)
$w^* = 1.35 * wdl =$	0.33 kN/m	$M^* =$	0.30 kNm (Max at 1350mm)
$w.dn^* = 1.2 * wdl + wwll^* =$	-0.29 kN/m (down)	$Mw.dn^* =$	0.00 kNm
$p(dl)^* = 1.35 * pdl =$	0.00 kN	$V(dl)^* =$	0.45 kN
$p^* = 1.2 * pdl + 1.5 * pll =$	0.00 kN	$V^* =$	0.45 kN
$p.dn^* = 1.2 * pdl + pwl^* =$	0.00 kN	$Vw.dn^* =$	0.40 kN

**Bending and Shear capacity - Cl 3.2.1 & Cl 3.2.5**

Member =	90mm x 45mm MGP10	Area (A) =	4050 mm <sup>2</sup>
Description =	MGP10 seasoned softwood	Section modulus (Zx) =	61 x10 <sup>3</sup> mm <sup>3</sup>
Design depth (dD) =	90 mm	Stiffness (Ix) =	2.7 x10 <sup>6</sup> mm <sup>4</sup>
Design width (dW) =	45 mm	Modulus of elasticity (E) =	10000 MPa - Table H3.1
$S1d = 1.25 * dD / dW^* (Lay.t/dD)^{0.5} =$	7.91 For top comp. edge restrained - Cl 3.2.3.2(a)		
$S1ud = 1.25 * dD / dW^* (Lay.t/dD)^{0.5} =$	7.91 For top comp. edge restrained - Cl 3.2.3.2(a)		
$k12d =$	1.000 for $pbd * S1d \leq 10$ - Cl 3.2.4	$f'b =$	17.0 MPa
$k12 =$	1.000 for $pb * S1d \leq 10$ - Cl 3.2.4	$f's =$	2.6 MPa
$k12u =$	1.000 for $pbu * S1u \leq 10$ - Cl 3.2.4	Material constant (pbd) =	0.75 (rbd=0.25)
Strength reduction factor ( $\phi$ ) =	0.9 Table 2.1	Material constant (pb) =	0.75 (rb=0.25)
$\phi M(dl) = \phi * (k1=0.57) * k4 * k6 * k9 * k12d * f'b * Zx =$	0.53 kNm	Stress reversal (pbu) =	0.69 (rbu=1.00)
$\phi M = \phi * k1 * k4 * k6 * k9 * k12u * f'b * Zx =$	0.87 kNm	Duration factor (k1) =	0.94 (Live)
$\phi Mw = \phi * (k1=1) * k4 * k6 * k9 * k12u * f'b * Zx =$	0.93 kNm	Moisture factor (k4) =	1.00
$\phi V(dl) = \phi * (k1=0.57) * k4 * k6 * f's * (2/3 * A) =$	3.60 kN	Temp. factor (k6) =	1.00
$\phi V = \phi * k1 * k4 * k6 * f's * (2/3 * A) =$	5.94 kN	Sharing factor (k9) =	1.00
$\phi Vw = \phi * (k1=1) * k4 * k6 * f's * (2/3 * A) =$	6.32 kN	Size modifier (mod.b & s) =	1.00 & 1.00

**Deflections**

		Duration factor ( $j2$ ) =	2.0
Ireq'd $j2.(DL+\Psi.LL)$ (L/300) =	3.8 x10 <sup>6</sup> mm <sup>4</sup>	$\delta DL =$	12.4 mm
Ireq'd $\Psi.LL$ (L/240) =	0.0 x10 <sup>6</sup> mm <sup>4</sup>	$\delta \Psi s.LL =$	0.0 mm
Ireq'd Ws (L/180) =	0.0 x10 <sup>6</sup> mm <sup>4</sup>	$\delta WLs =$	0.0 mm
			Span / 217
			Span / 228571
			Span / -

**PORTAL KNEE CONNECTION V5.08**

**Rafter forces (At the face of the column) (Knee Connection PK01)**

Rafter bending moment ( $M^*$ ) =	270 kNm	(+ve = top flange in tension ie. DL + LL)
Rafter axial force ( $Nt^*$ ) =	50 kN	(+ve = tension ie. WL)
Rafter shear force ( $V^*$ ) =	35 kN	(+ve = acting downward ie. DL + LL)

**Column forces (At the underside of the Rafter / Haunch)**

Column moment  $M_c^*$  does not reduce web capacity

Column bending moment ( $M_c^*$ ) =	270 kNm	(+ve = outside flange in tension ie. DL + LL)
Column shear force ( $V_c^*$ ) =	35 kN	(+ve = acting in same dir. - typ. DL + LL)

**Member Design Capacities**

Rafter member capacity ( $\phi M_{br,r}$ ) =	448.2 kNm
Rafter section capacity ( $\phi M_{sx,r}$ ) =	448.2 kNm
Column section capacity ( $\phi M_{sx,c}$ ) =	639.9 kNm

**Minimum design actions - Cl 9.1.4 of AS4100**

Min. design rafter moment ( $M_a = \text{Max}(\phi M_{br,x}, M^*)$ ) =	270.0 kNm	Ignore min. rafter $M^* =$	N (Y)es,(N)o
Min. design shear ( $V_a^* = \text{Max}(\text{Min}(40\text{kN}, 0.15 \cdot \phi V_{vm}), V^*)$ ) =	40.0 kN		
Design bending moment ( $M_d^*$ ) =	270.0 kNm		
Design axial force ( $N_d^*$ ) =	50.0 kN		
Design shear force ( $V_d^*$ ) =	40.0 kN		

**Calculated forces in the connection - Table 4.8.2.2**

Top flange in tension from rafter moment

Design tension flange force ( $N_{ft}^*$ ) =	635.3 kN
Design comp. flange force ( $N_{fc}^*$ ) =	583.3 kN
Design shear force at End Plate/Column ( $V_{vc}^*$ ) =	37.3 kN

Stress regions overlap = No      Cl 4.8.3.4(c) - For stiffener recommendation

**TIMBER JOINTS V5.02 - Nails**

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Member:	(Timber Joint TJ01 FJ1) Smart LVL13 (JD4) with 20/3.15mm Dia. Nails	Type 1 or 2 only	Group
Shear:	$V^* = 7.66\text{kN} < \phi N_{d,jl} = 8.55\text{kN}$	OK (0.90)	OK (0.90)
Moment:	$M^* = 0.00\text{kNm} < \phi N_{d,jm} = 0.26\text{kNm}$	OK (0.00)	
Pullout:	$N_p^* = 0.00\text{kN} < \phi N_{d,ja} = 5.95\text{kN}$	OK (0.00)	OK (0.00)
Combined:			OK (0.90)

**Loading** (House member affecting  $\leq 25\text{m}^2$  or secondary building member)

Category = 1 (1) House, (2) Primary building elements, (3) Important  
Live load duration = Floor  
Load duration factor ( $k_1$ ) = 0.69 Table 2.3

**Type 1 - 1.2\*G+1.5\*Q**

Design axial ( $N_t^*$ ) = 0.00 kN (Tension +ve, parallel to grain)  
Design shear ( $N_v^*$ ) = 7.66 kN (Shear down +ve, perpendicular to grain)  
Resolved ( $N^*$ ) = 7.66 kN

Joint moment ( $M^*$ ) = 0.00 kNm

**Type 2 - 1.2\*G+1.5\*Q**

Pullout Force ( $N_p^*$ ) = 0.00 kN (Out-of-plane)

**Geometry**

**Fixing geometry**

Nail diameter (D) = 3.15 mm  
Nail rows = 10  
Nail columns = 2  
No. of nails = 20

Nails driven = S (S)traight/s(K)ew  
Drive method = H (H)and,(G)un

Fixing in side grain = Y (Y)es,(N)o  
Single or double shear = S (S)ingle,(D)ouble  
Side plates = N (M)etal,(P)lywood,(N)one  
Close fitting holes = Y (Y)es,(N)o  
Holes pre-drilled = Y (Y)es,(N)o - to 80% nail diameter

**Member geometry**

Thickness of first member ( $t_1$ ) = 45 mm      Min.  $t_1$  = 15.8 mm (31.5mm full effect.)  
Length of penetration into second member ( $l_p=tp$ ) = 35 mm      Min.  $tp$  = 15.8 mm (31.5mm full effect.)  
Connection depth across grain = 117.1 mm      Min. depth = 117.1 mm  
Input geometry depth = 117.1 mm

**Nail spacing - Table 4.4**

End distance (parallel) = 31.5 mm      Min. end = 31.5 mm  
Edge distance (perpendicular) = 15.8 mm      Min. edge = 15.8 mm  
Centres along grain (parallel) = 31.5 mm      Min. along = 31.5 mm  
Centres across grain (perpendicular) = 9.5 mm      Min. across = 9.5 mm

**Minimum nail spacing - Table 4.4**

End distance (parallel/along grain) = 10D = 31.5 mm      (Pre-drilled holes)  
Edge distance (perpendicular/across grain) = 5D = 15.8 mm  
Spacing (parallel/along grain) = 10D = 31.5 mm  
Spacing (perpendicular/across grain) = 3D = 9.5 mm

**TIMBER JOINTS V5.02 - Nails**

Deery Consulting Structural Engineer

**Material grade**

Material = Smart LVL13  
Strength Group = SD6 (SD6)  
Joint Group = JD4 (JD4)  
Seasoned = Y (Yes),(N)o

**Material thickness - 2 members**

Nail shear = Single shear

Thickness of 1st memb. for full effect. (t1f) = 10*D =	31.5 mm - Cl 4.2.5(a)	
Thickness of 1st memb. for min. effect. (t1m) = 5*D =	15.8 mm - Cl 4.2.5(a)	
Pen. into 2nd memb. for full effect. (tpf) = 10*D =	31.5 mm - Cl 4.2.5(a)	
Pen. into 2nd memb. for min. effect. (tpm) = 5*D =	15.8 mm - Cl 4.2.5(a)	
Thickness of first member (t1) =	45.0 mm	Fully effective for Qk Table 4.1(A)&(B)
Length of penetration into second member (lp=tp) =	35.0 mm	Fully effective for Qk Table 4.1(A)&(B)

**Type 1 joint to resist direct loads (Without moment) - Cl 4.2.2.2 & Cl 4.2.3.2**

Fastener strength reduction factor ( $\phi_f$ ) =	0.85 Table 2.2	
Load duration factor ( $k_1$ ) =	0.69 Table 2.3	
Side grain ( $k_{13}$ ) =	1.00 (Side grain) - Cl 4.2.2.2	
Single shear ( $k_{14}$ ) =	1.00 (Single shear)	
Side plates ( $k_{16}$ ) =	1.00 (None)	
Multiple fixings group (direct) ( $k_{17}$ ) =	0.90 (10 rows of nails)	
Total fixings resisting (n) =	20	
Reduction (red) = $\min(1, t_1/t_{1f}, t_p/tpf)$ =	1.000	
Qkl =	810 N - Table 4.1(B)	
$\phi_{Nd, j1} = \phi_f * k_1 * k_{13} * k_{14} * k_{16} * k_{17} * n * red * Qkl$ =	8551 N	
$\phi_{Nd, j1}$ =	8.55 kN	OK (0.90)

**Type 1 joint to resist direct loads (Moment) - Cl 4.2.2.2 & Cl 4.2.3.3**

Fastener strength reduction factor ( $\phi_f$ ) =	0.85 Table 2.2	
Multiple nailed group (moment) ( $k_{17}$ ) =	1.08 (8 effective fixings)	
Total fixings resisting (n) =	20	
rmax (rm.) =	45.6 mm	
$\Sigma r_i / r_m.^{3/2}$ =	11.02	
Qkl =	810 N - Table 4.1(B)	
$\phi_{Md, j1} = \phi_f * k_1 * k_{13} * k_{14} * k_{16} * k_{17} * r_m. * re. * Qkl * \Sigma r_i / r_m.^{3/2}$ =	257659 Nmm	
$\phi_{Md, j1}$ =	0.26 kNm	OK (0.00)

**Type 2 joint (Pullout/Withdrawal) - Cl 4.2.2.3 & Cl 4.2.3.4**

Nails gun driven in end grain may have significantly reduced capacities than hand driven - Cl 4.2.2.3(a)

Fastener strength reduction factor ( $\phi_f$ ) =	0.85	
Side grain ( $k_{13w}$ ) =	1.00 (Side grain) - Cl 4.2.2.3	
Length of penetration (lp = tp) =	35 mm	
Total fixings resisting (n) =	20	
Gun driven reduction (gred) =	1	
Reduction (red) =	1 (0 if less than 2 nails driven into end grain)	
Qka =	10 N - Table 4.2(B)	
$\phi_{Nd, ja} = \phi_f * k_{13w} * l_p * n * red * gred * Qka$ =	5950 N	
$\phi_{Nd, ja}$ =	5.95 kN	OK (0.00)

**TIMBER JOINTS V5.02 - Nails**

Deery Consulting Structural Engineer

**Nail layout**

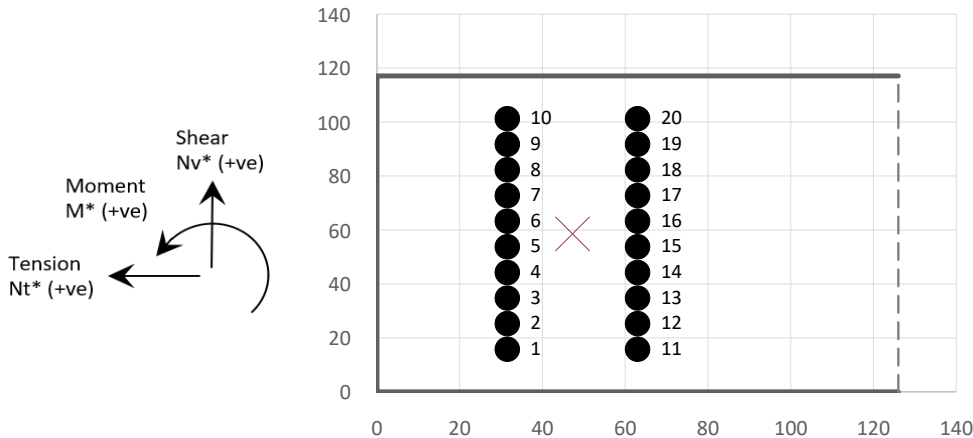
Nail	Row #	Column #	x mm	y mm	Ratio $\Sigma F^*/\phi Nd.jl$	Ratio $\Sigma Tz^*/\phi Nd.ja$	Ratio $(\Sigma F^*/\phi Nd.jl)^2 + (\Sigma Tz^*/\phi Nd.ja)^2$	Valid
1	1	1	31.5	15.8	0.896	0.000	0.896	1
2	2	1	31.5	25.3	0.896	0.000	0.896	1
3	3	1	31.5	34.8	0.896	0.000	0.896	1
4	4	1	31.5	44.3	0.896	0.000	0.896	1
5	5	1	31.5	53.8	0.896	0.000	0.896	1
6	6	1	31.5	63.3	0.896	0.000	0.896	1
7	7	1	31.5	72.8	0.896	0.000	0.896	1
8	8	1	31.5	82.3	0.896	0.000	0.896	1
9	9	1	31.5	91.8	0.896	0.000	0.896	1
10	10	1	31.5	101.3	0.896	0.000	0.896	1
11	1	2	63	15.8	0.896	0.000	0.896	1
12	2	2	63	25.3	0.896	0.000	0.896	1
13	3	2	63	34.8	0.896	0.000	0.896	1
14	4	2	63	44.3	0.896	0.000	0.896	1
15	5	2	63	53.8	0.896	0.000	0.896	1
16	6	2	63	63.3	0.896	0.000	0.896	1
17	7	2	63	72.8	0.896	0.000	0.896	1
18	8	2	63	82.3	0.896	0.000	0.896	1
19	9	2	63	91.8	0.896	0.000	0.896	1
20	10	2	63	101.3	0.896	0.000	0.896	1
21								
22								
23								
24								
25								
<b>Max =</b>					0.896	0.000	0.896	20

	x mm	y mm	$\Sigma lx$ mm <sup>2</sup>	$\Sigma ly$ mm <sup>2</sup>	$\Sigma lx + \Sigma ly$ mm <sup>2</sup>
Centroid =	47.3	58.6	4961.3	14891.3	19852.5

Number of nails resisting (n) = 20  
 Number rows (nr) = 10  
 Columns (nc) = 2  
 Effective fixings for k17 = 8

Min. member length (parallel) = 94.5 mm  
 Min. member depth (perp.) = 117 mm

rmax = 45.6 mm  
 $\Sigma ri/rmax^{3/2} = 11.02$



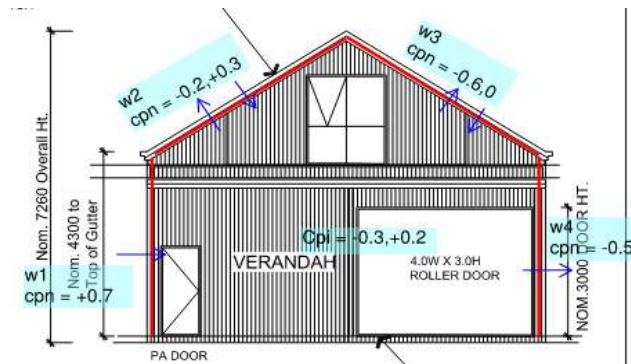
Licensee: Deery Consulting Structural Engineer

## PORTAL FRAME SPACEGASS LOADS (AS 1170.2)

P (kPa)	1.3
Lw (m)	5.8

h (m)	5.7
b (m)	12
d (m)	9

h/d	0.64
d/b	0.75
$\alpha$	32



## ROOF LOADS

Roof	Load (kPa)	kN/m	Direction
Dead Load Min	-0.12	-0.7	↓
Dead Load Max	-0.25	-1.45	↓
Live Load	-0.25	-1.45	↓

## WIND LOADS

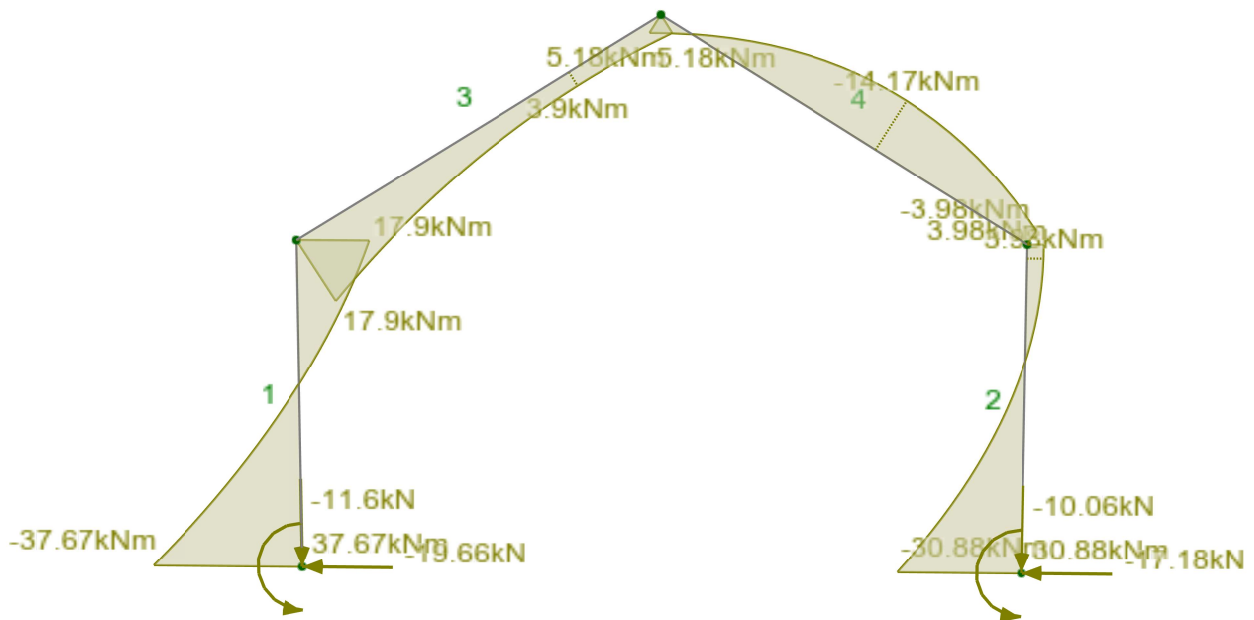
Cross Wind Up	Cpe	Cpi	Cpi,Cpe	Kci,Kce	kN/m	Direction
W1	0.7	0.2	0.5	0.8	3.02	→
W2	-0.2	0.2	-0.4	0.8	-2.42	↖
W3	-0.6	0.2	-0.8	0.8	-4.83	↗
W4	-0.5	0.2	-0.7	0.8	-4.23	→
W5			0	0.8	0	
W6			0	0.8	0	

Cross Wind Down	Cpe	Cpi	Cpi,Cpe	Kci,Kce	kN/m	Direction
W1	0.7	-0.3	1	0.8	6.04	→
W2	0.3	-0.3	0.6	0.8	3.62	↘
W3	0	-0.3	0.3	0.8	1.81	↙
W4	-0.5	-0.3	-0.2	0.8	-1.21	→
W5			0	0.8	0	
W6			0	0.8	0	

SPACE GASS 14.11 - DEERY CONSULTING STRUCTURAL & CIVIL ENGINEERS  
 Path: G:\shortcut-targets-by-id\lkyxyi7fTTxVbaxU1J...\Main Frame\SG - 17457  
 Designer: Date: Friday, December 20, 2024 10:33 AM, Page: 1  
 Filter: Fixed base



Load case 11  
 11 (SW) 0.9DL+Wind UP



Viewpoint (2,11), Moments, Reactions

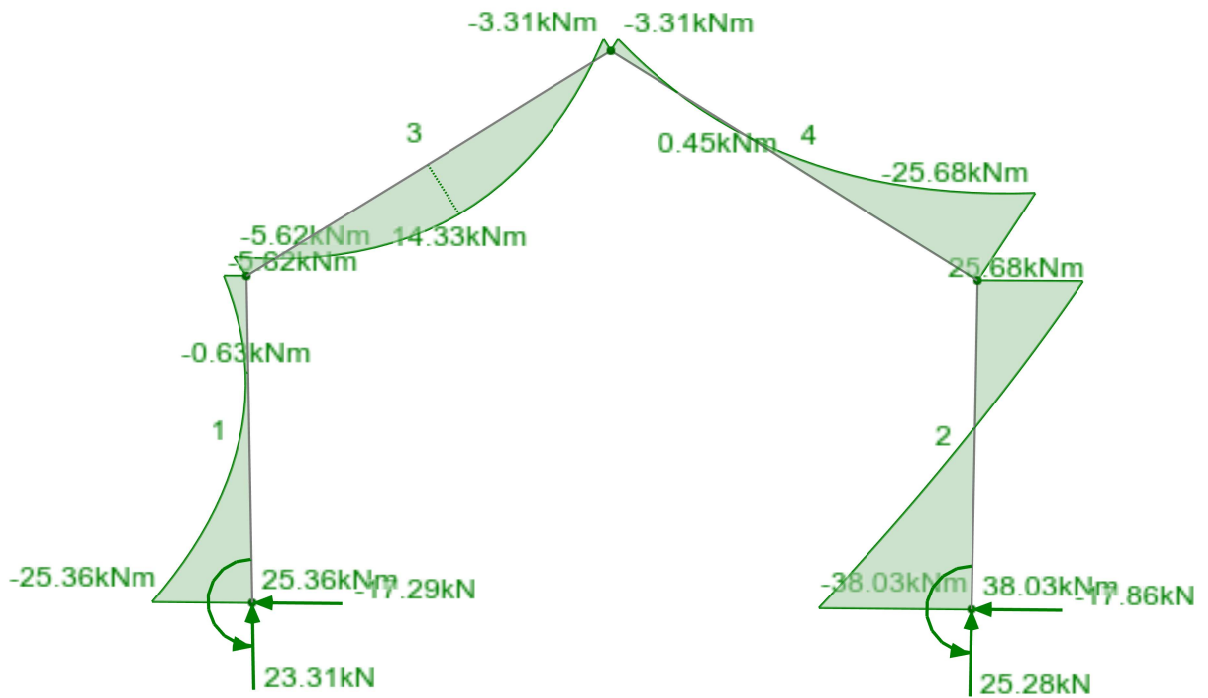
Materials: Sections:  
 1 STEEL 1 250 UB 25.7

SPACE GASS 14.11 - DEERY CONSULTING STRUCTURAL & CIVIL ENGINEERS  
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 Designer: Date: Friday, December 20, 2024 10:33 AM, Page: 1  
 Filter: Fixed base



Load case 12

■ 12 (SW) 1.2DL+Wind DOWN



Viewpoint (2,11), Moments, Reactions

Materials:

■ 1 STEEL

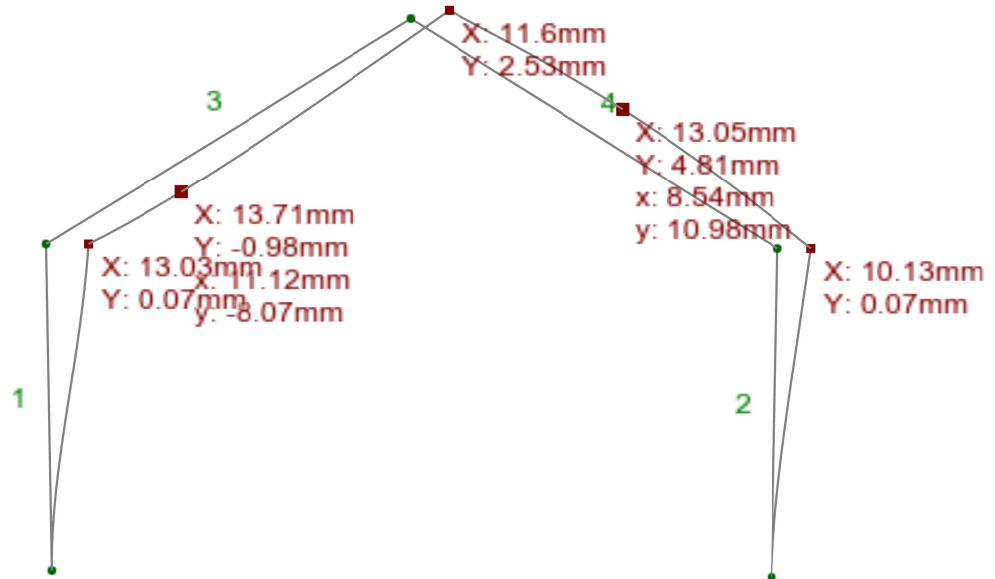
Sections:

■ 1 250 UB 25.7

SPACE GASS 14.11 - DEERY CONSULTING STRUCTURAL & CIVIL ENGINEERS  
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Designer: Date: Friday, December 20, 2024 10:34 AM, Page: 1  
Filter: Fixed base



Load case 20  
■ 20 Service Wind UP



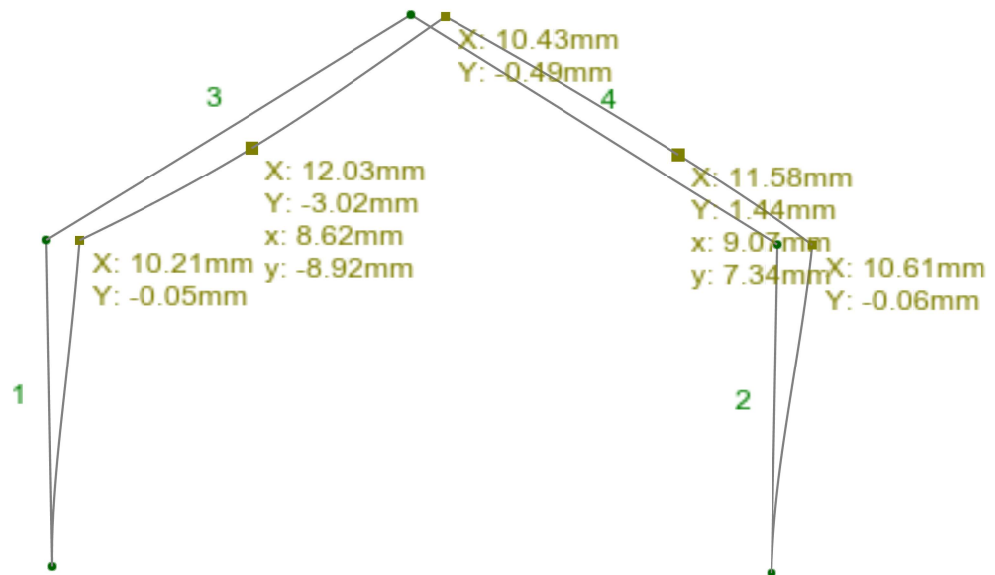
Viewpoint (2,11), Displacements

Materials: 1 STEEL  
Sections: 1 250 UB 25.7

SPACE GASS 14.11 - DEERY CONSULTING STRUCTURAL & CIVIL ENGINEERS  
Path: G:\shortcut-targets-by-id\lkyxyi7fTTxVbaXUJ...\Main Frame\SG - 17457  
Designer: Date: Friday, December 20, 2024 10:34 AM, Page: 1  
Filter: Fixed base



Load case 21  
■ 21 Service Wind DOWN



Viewpoint (2,11), Displacements

Materials:

■ 1 STEEL

Sections:

■ 1 250 UB 25.7