

# **Prime Consulting Engineers Pty. Ltd.**

# **Design Report:**

# 3.5m Square Cantilever Umbrella

For



Ref: R-22-174-2

Date: 20/01/2022

Amendment: -

Prepared by: KZ

Checked by: BG

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## 1 Introduction and Scope:

The report and certification are the sole property of Prime Consulting Engineers Pty. Ltd.

Prime Consulting Engineers have been engaged by Extreme Marquees Pty. Ltd. to carry out a structural analysis of three different sizes of Aluminium Cantilever Umbrellas for wind region A (non-cyclonic). It should be noted that the outcome of our analysis is limited to the selected items as outlined in this report.

This report shall be read in conjunction with the documents listed in the references (Section 1.2)

### 1.1 Project Description

The report examines the effect of 3s gust wind of **(refer to summary)** positioned for the worst effect on 3.5m square cantilever umbrella structure. The relevant Australian Standards AS1170.0:2002 General principles, AS1170.1:2002 Permanent, imposed and other actions and AS1170.2:2011 Wind actions are used. The design check is in accordance with AS1664.1 Aluminum Structures.

#### 1.2 References

- The documents referred to in this report are as follows:
  - Report of results produced through SAP2000 V23 software & excel spreadsheets.
  - Detail drawing provided by manufacturer (YEEZE). Refer to appendix 'A'.
- The basic standards used in this report are as follows:
  - AS 1170.0:2002 Structural Design Actions (Part 0: General principles)
  - AS 1170.1:2002 Structural Design Actions (Part 1: Permanent, imposed, and other actions)
  - AS 1170.2:2011 Structural Design Actions (Part 2: Wind Actions)
  - AS1664.1 Aluminium Structures.
- Section Properties of Aluminium Section provided by the client. (Refer Appendix 'A'.
- The program(s) used for this analysis are as follows:
  - o SAP2000 V23
  - Microsoft Excel

#### 1.3 Notation

AS/NZS Australian Standard/New Zealand Standard

FEM/FEA Finite Element Method/Finite Element Analysis

SLS Serviceability Limit State

ULS Ultimate Limit State

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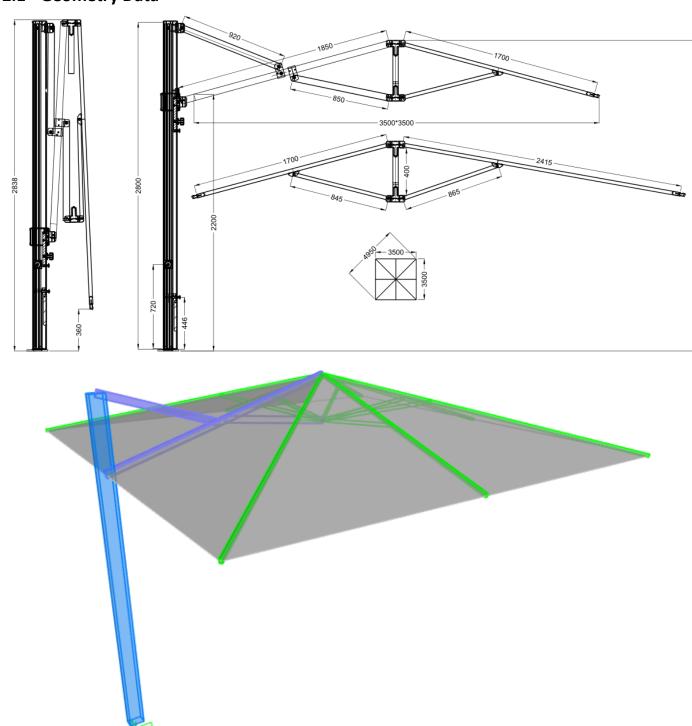
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# 2 Design Overview

## 2.1 Geometry Data



Isometric view of structures

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### 2.2 Assumptions & Limitations

- The erected structure is for temporary use only.
- For forecast winds in excess of (refer to summary) the umbrella structure should be completely folded
- The structure may only be used in regions with wind classifications no greater than the limits specified in cl. 5 of this report.
- Parameters used for wind calculations:
  - TC 2
  - Wind Region A
- Topographical factors such as erecting the structure on the crest of a hill or on the top of an escarpment may result in a higher wind speed classification. Thus, special considerations should be taken to the topographical location of the installation site.
- Shall the site conditions/wind parameters exceed prescribed design wind actions (refer to cl.8), Prime Consulting Engineers Pty. Ltd. should be informed to determine appropriate wind classifications and amend computations accordingly.

#### 2.3 Exclusions

- Design of fabric
- Wind actions due to tropical or severe tropical cyclonic areas.
- Super imposed loads such as live loads or snow and ice loads.

### 2.4 Design Parameters and Inputs

#### 2.4.1 Load Cases

G Permanent actions (Dead load)
 Wu Ultimate wind action (ULS)
 Ws Serviceability wind action (SLS)

#### 2.4.2 Load Combinations

#### Strength (ULS):

1. 1.35G Permanent action only
 3. 0.9G+W<sub>u</sub> Permanent and wind actions
 4. 1.2G+W<sub>u</sub> Permanent and wind actions

Serviceability (SLS):

2. G+W<sub>s</sub> Wind service actions

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# 3 Specifications

## 3.1 Material Properties

Material Properties												
COCO TE	Ftu	Fty	Fcy	Fsu	Fsy	F <sub>bu</sub>	F <sub>by</sub>	Е	kt	<b>k</b> c		
6063-T5	152	110	110	90	62	317	179	70000	1	1.12		

## 3.2 Buckling Constants

	TABLE 3.3(D) BUCKLING CONSTANTS												
Type of member and stress	Interce	ept, MPa		ope, IPa	Intersection								
Compression in columns and beam flanges	B <sub>c</sub>	119.26	D <sub>c</sub>	0.49	Cc	99.33							
Compression in flat plates	Bp	134.29	Dp	0.59	Cp	93.61							
Compression in round tubes under axial end load	Bt	132.00	Dt	3.62	Ct	*							
Compressive bending stress in rectangular bars	B <sub>br</sub>	194.52	D <sub>br</sub>	1.26	C <sub>br</sub>	103.26							
Compressive bending stress in round tubes	B <sub>tb</sub>	183.09	D <sub>tb</sub>	9.34	Ctb	79.80							
Shear stress in flat plates	Bs	75.86	Ds	0.25	Cs	124.54							
Ultimate strength of flat plates in compression	<b>k</b> <sub>1</sub>	0.35	k <sub>2</sub>	2.27									
Ultimate strength of flat plates in bending	<b>K</b> <sub>1</sub>	0.5	k <sub>2</sub>	2.04									

 $<sup>^*</sup>$   $C_t$  shall be determined using a plot of curves of limit state stress based on elastic and inelastic buckling or by trial and error solution

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## 3.3 Member Sizes & Section Properties

## 3.3.1 Rectangular Section

MEMBER(S)	Section	b	d	t	у <sub>с</sub>	Ag	Z <sub>x</sub>	Z <sub>y</sub>	S <sub>x</sub>	Sy	l <sub>x</sub>	l <sub>y</sub>	J	r <sub>x</sub>	r <sub>y</sub>
		mm	mm	mm	mm	mm²	mm³	mm³	mm³	mm³	mm⁴	mm⁴	mm⁴	mm	mm
Post	120x85x3	85	120	3	60.0	1194.0	41441.7	34291.3	49329.0	38881.5	2486502.0	1457379.5	2775221.2	45.6	34.9
Cantilever Beam	60x35x3.5	35	60	3.5	30.0	616.0	9420.7	6709.7	11837.0	7987.0	282620.3	117420.3	251961.0	21.4	13.8
Brace 1	60x35x3.5	35	60	3.5	30.0	616.0	9420.7	6709.7	11837.0	7987.0	282620.3	117420.3	251961.0	21.4	13.8
Brace 2	30x20x1.5	20	30	1.5	15.0	141.0	1141.1	894.6	1401.8	1049.3	17115.8	8945.8	17744.2	11.0	8.0
Middle Beam	30x20x1.5	20	30	1.5	15.0	141.0	1141.1	894.6	1401.8	1049.3	17115.8	8945.8	17744.2	11.0	8.0
Corner Beam	30x20x1.5	20	30	1.5	15.0	141.0	1141.1	894.6	1401.8	1049.3	17115.8	8945.8	17744.2	11.0	8.0
Brace	100x50x5	50	100	5	50.0	1400.0	34733.3	22466.7	44000.0	26500.0	1736666.7	561666.7	1305401.8	35.2	20.0

### 3.3.2 Circular Sections

MEMBER(S)	Section	đ	t	Уc	$\mathbf{A}_{\mathrm{g}}$	Z <sub>x</sub>	Z <sub>y</sub>	S <sub>x</sub>	Sy	l <sub>x</sub>	l <sub>y</sub>	J	r <sub>x</sub>	r <sub>y</sub>
		mm	mm	mm	mm²	mm³	mm³	mm³	mm³	mm⁴	mm⁴	mm⁴	mm	mm
Centre Pole	48x1.8	48	1.8	24.0	261.3	2908.7	2908.7	3843.9	3843.9	69809.9	69809.9	139619.8	16.3	16.3

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# 4 Design Loads

Self weight	G	self weight
3s 55km/hr gust	Wu	0.116 C <sub>fig</sub> (kPa)
3s 20km/hr gust	Ws	0.015 C <sub>fig</sub> (kPa)

# 5 Wind Analysis

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## 5.1 Ultimate

Project: 4m square Cantilever Umbrella

Job no. 22-174-2 Designer: KZ

Name	Symbol	Value	Unit	Notes	Ref.
		Inp	ut		
Importance level		2			Table 3.1 - Table 3.2 (AS1170.0)
Annual probability of exceedance		Temporary			Table 3.3
Regional gust wind speed		55.008	Km/hr		
Regional gust wind speed	$V_{R}$	15.28	m/s		
Wind Direction Multipliers	$M_{\text{d}}$	1			Table 3.2 (AS1170.2)
Terrain Category	TC	2			, ,
Terrain Category Multiplier	$M_{Z,Cat}$	0.91			
Shield Multiplier	Ms	1			4.3 (AS1170.2)
Topographic Multiplier	$M_{t}$	1			4.4 (AS1170.2)
Site Wind Speed	$V_{Site,\beta}$	13.90	m/s	$V_{Site,\beta}=V_R*M_d*M_{z,cat}*M_S,M_t$	
Pitch	α	15	Deg		
Pitch	α	-	rad		

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Width	В	4	m								
Length	D	4	m								
Height	Z	2.5	m								
Porosity Ratio	δ	1		ratio of solid area to total area							
				arou							
		Wind P	Pressure								
hoair	$\rho$	1.2	Kg/m <sup>3</sup>								
dynamic response factor	$C_dyn$	1									
Wind Pressure	ho*Cfig	0.116	Kg/m <sup>2</sup>	$\rho$ =0.5 $\rho$ air*( $V$ des, $\beta$ ) <sup>2</sup> * $C$ fig* $C$ dyn	2.4 (AS1170.2)						
		WIND DIREC	CTION 1 (6	 Э=0)							
External Pressure											
1. Free Roof				α <b>=0</b> °							
Area Reduction Factor	Ka	1			D7						
local pressure factor	Κı	1									
porous cladding reduction factor	$K_p$	1.00									
External Pressure Coefficient MIN	$C_{P,w}$	-0.3									
External Pressure Coefficient MAX	$C_{P,w}$	0.4									
External Pressure Coefficient MIN	$C_{P,I}$	-0.4									
External Pressure Coefficient	$C_{P,I}$	0									
MAX aerodynamic shape factor	$C_{fig,w}$	-0.30									
MIN aerodynamic shape factor	C <sub>fig,w</sub>	0.40									
MAX aerodynamic shape factor	C <sub>fig,l</sub>	-0.40									
MIN aerodynamic shape factor	-										
MAX	$C_{\mathrm{fig,I}}$	0.00									
Pressure Windward MIN	Р	-0.03	kPa								
Pressure Windward MAX	Р	0.05	kPa								
Pressure Leeward MIN	Р	-0.05	kPa								
Pressure Leeward MAX	Р	0.00	kPa								
		WIND DIREC									
		External	Pressure								
4. Free Roof				α <b>=180°</b>	D7						

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Area Reduction Factor	Ka	1	
local pressure factor	$K_{l}$	1	
porous cladding reduction factor	$K_p$	1.00	
External Pressure Coefficient MIN	$C_{P,w}$	-0.3	
External Pressure Coefficient <b>MAX</b>	$C_{P,w}$	0.4	
External Pressure Coefficient MIN	$C_{P,I}$	-0.4	
External Pressure Coefficient <b>MAX</b>	$C_{P,I}$	0	
aerodynamic shape factor MIN	$C_{\text{fig},w}$	-0.30	
aerodynamic shape factor <b>MAX</b>	$C_{\text{fig},w}$	0.40	
aerodynamic shape factor MIN	$C_{\text{fig,I}}$	-0.40	
aerodynamic shape factor <b>MAX</b>	$C_{\text{fig,I}}$	0.00	
Pressure MIN (Windward	Р	0.00	l-D-
Side)	۲	-0.03	kPa
Pressure MAX (Windward Side)	Р	0.05	kPa
Pressure MIN (Leeward Side)	Р	-0.05	kPa
Pressure MAX (Leeward Side)	Р	0.00	kPa

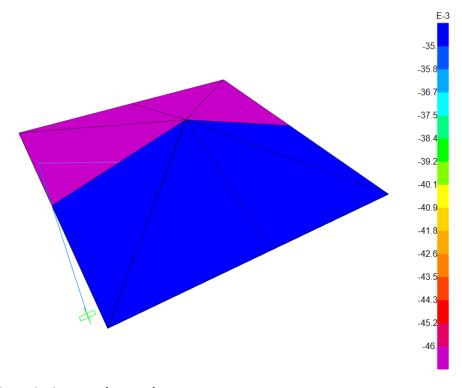
WIND EXTERNAL PRESSURE	Dire	ction1	Direction2		
WIND EXTERNAL PRESSURE	Min (Kpa)	Max (Kpa)	Min (Kpa)	Max (Kpa)	
Windward	-0.035	0.046	-0.035	0.046	
Leeward	-0.046	0.000	-0.046	0.000	

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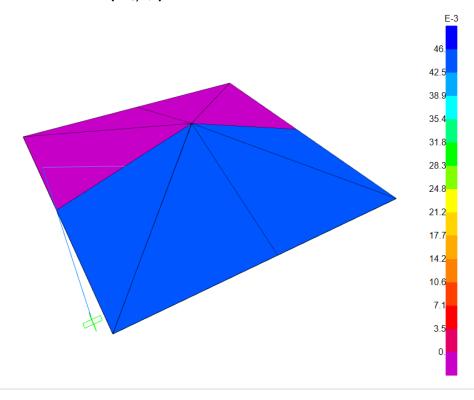


## 5.2 Load Diagrams

## 5.2.1 Wind Load Ultimate (W<sub>U,min</sub>)



## 5.2.2 Wind Load Ultimate (W<sub>U,max</sub>)



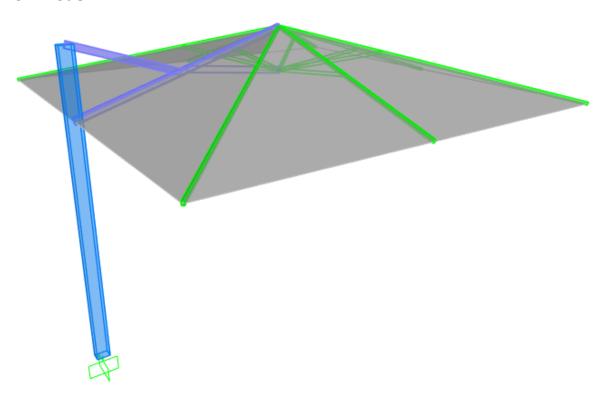
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# 6 Analysis

## **6.1** 3D model

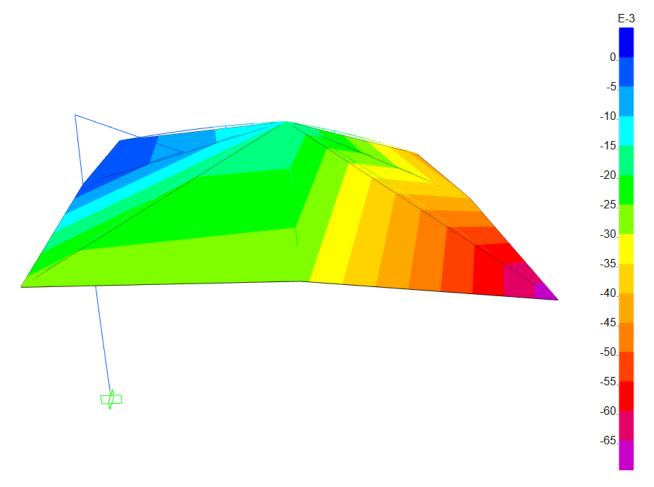


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## 6.2 Results

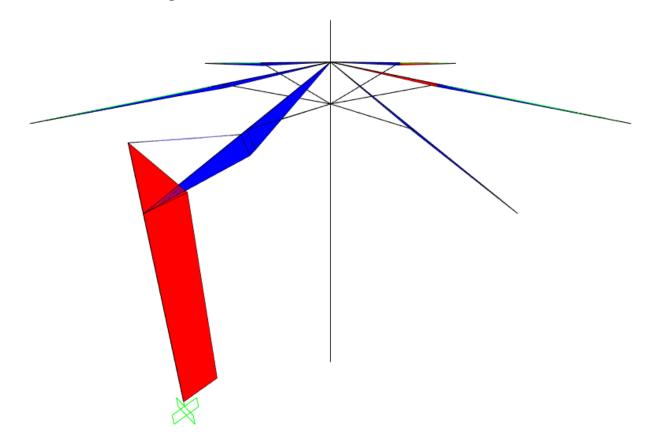
## 6.2.1 Maximum deflection (serviceability)



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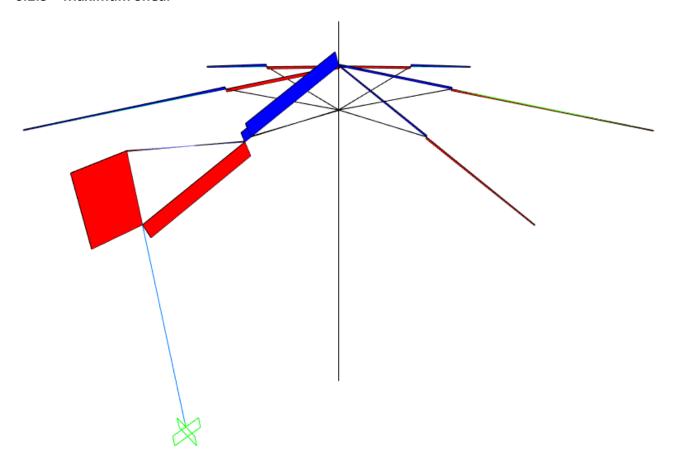


## 6.2.2 Maximum Bending Moment





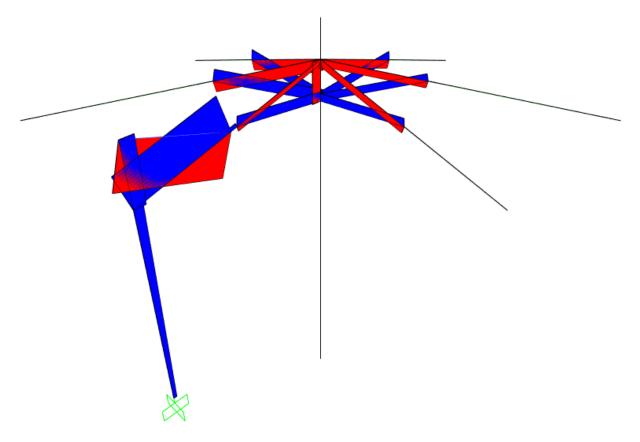
### 6.2.3 Maximum Shear



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### 6.2.4 Maximum Axial Force



#### 6.2.5 Maximum Reactions

	TABLE: Joint Reactions											
F1 F2 F3 M1 M2 M3												
OutputCase	KN	KN	KN	KN-m	KN-m	KN-m						
1.2G+Wmax	1.964E-12	-0.048	0.608	-0.1317	-0.8242	-0.0845						
0.9G+Wmin	-1.55E-12	-0.012	-0.251	-0.0315	0.6199	-0.0202						

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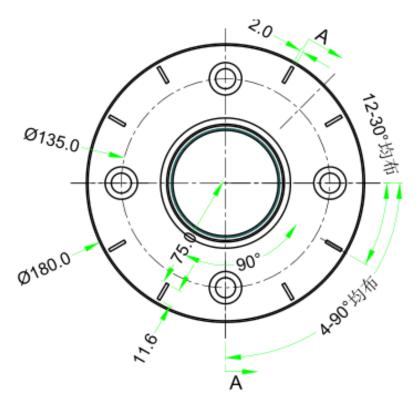
## 7 Aluminium Design

All members pass for the defined design wind actions. Refer to Appendix 'B' for section capacities and factor of safeties.

## 8 Anchorage Design

### 8.1 Bolted Structure

Refer to Appendix 'C' for details.



Base Plate Radius: 90mm Edge distance: 25mm

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Assumed Concrete Slab Thickness: 180mm Maximum Tensile Force on bolts: 5.33kN Design of supporting concrete slab is by others.

Use 4/HLA-Z1 M10 bolt by All Fasteners

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## 8.2 Weighted structure



Base Plate Holder: 850mm x 850mm x 70mm

Design forces:

 $M^* = 0.62 \text{ kN.m}$ P = 0.25 kN

 $0.73 \times 0.85 + 0.251 \times 0.85/2 = W/2 \times 0.85 \rightarrow W = 1.71kN$ 

180kg ballast is required to be distributed evenly on the 850 x 850 x 70 base plate holder

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## 9 Summary and Recommendations

 The 3.5m Square Cantilever Umbrella Structure as specified is capable of withstanding 3s gust wind speed up to <u>55km/hr</u>.

The umbrella structure is required to be folded for forecast winds in excess of <u>20km/hr</u> to avoid any potential permanent deformation/buckling due to excessive deflection as a result of higher wind speeds.

 The anchorage system described in <u>Cl. 8</u> (180kg ballast or 4/HLA-Z1 M10 bolt) is required to resist against uplift & overturning forces due to design wind loads.

Yours faithfully,

Prime Consulting Engineers Pty. Ltd.

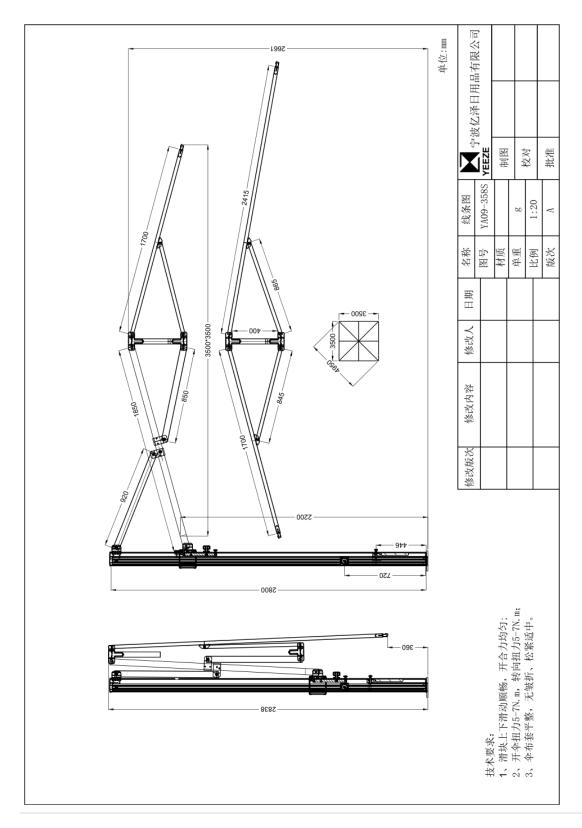
Kevin Zia, BEng, Meng, MIEAust, CPENG NER

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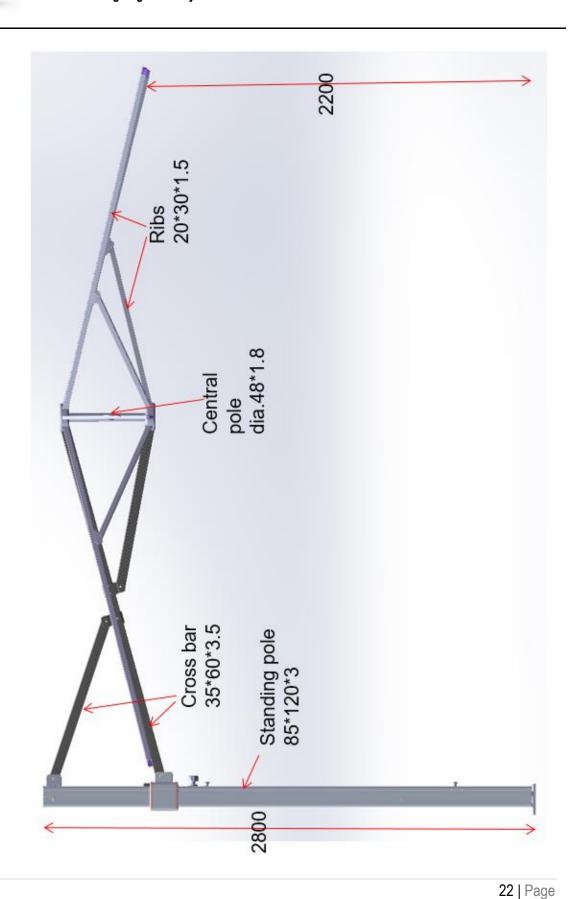


# 10 Appendix A – Detail Drawings



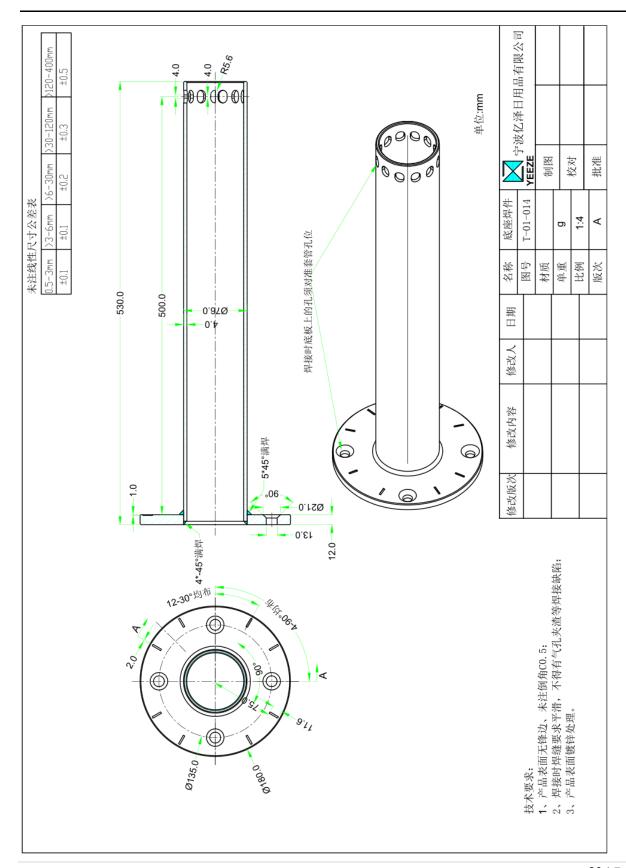
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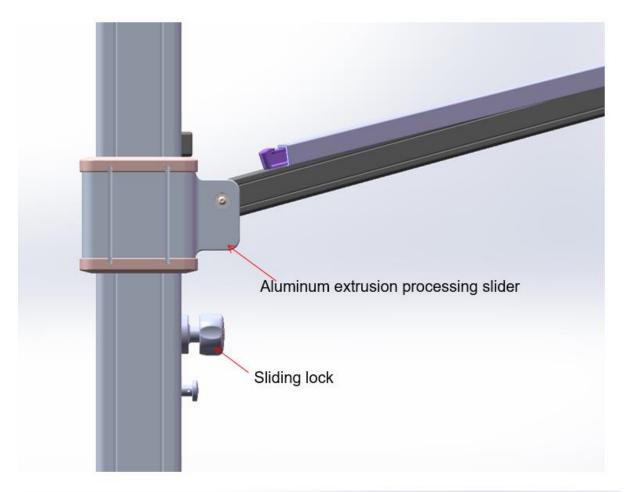


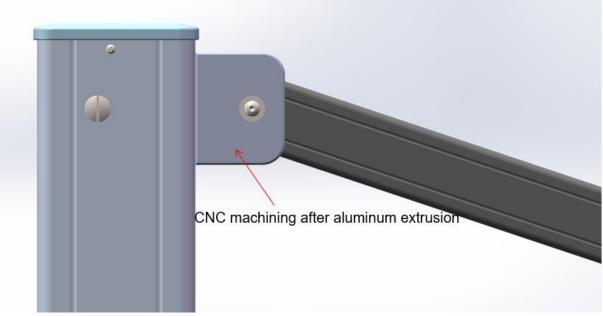
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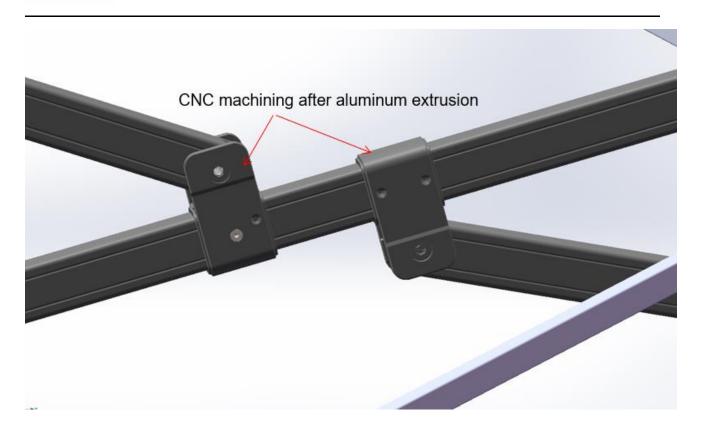






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# 11 Appendix B – Section capacity

## 11.1 Checking Members Based on AS1664.1 ALUMINIUM LSD

### 11.1.1 Post



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NAME	SYMBOL		VALUE	UNIT	NOTES	REF
120x85x3	Post					
Alloy and temper	6063-T5					AS1664.1
	Ftu	=	152	MPa	Ultimate	T3.3(A)
Tension	F <sub>ty</sub>	=	110	MPa	Yield	10.5(71)
Compression	F <sub>cy</sub>	_	110	MPa	Tiold	
Compression	F <sub>su</sub>	_	90	MPa	Ultimate	
Shear	F <sub>sy</sub>	=	62	MPa	Yield	
	F <sub>bu</sub>	=	317	MPa	Ultimate	
Bearing	F <sub>by</sub>	=	179	мРа	Yield	
	Гъу	_	179	IVIFA	rieid	
Modulus of elasticity	E	=	70000	MPa	Compressive	
•						
	$\mathbf{k}_{t}$	=	1			T3.4(B)
	<b>k</b> c	=	1			10.4(D)
FEM ANALYSIS RESULTS						
Axial force	Р	=	0.524	kN	compression	
	Р	=	0	kN	Tension	
In plane moment	$M_{x}$	=	0.8242	kNm		
Out of plane moment	$M_{y}$	=	0.2379	kNm		
·						
DESIGN STRESSES						
Gross cross section area	$A_g$	=	1194	mm <sup>2</sup>		
In-plane elastic section modulus	$Z_{x}$	=	41441.7	$\rm mm^3$		
Out-of-plane elastic section	7		24204 202	mm³		
mod.	$Z_y$	=	34291.282	mm <sup>3</sup>		
Stress from axial force	f <sub>a</sub>	=	P/A <sub>g</sub>			
		=	0.44	MPa	compression	

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		=	0.00	MPa	Tension	
Stress from in-plane bending	$f_{bx}$	=	$M_x/Z_x$			
		=	19.89	MPa	compression	
Stress from out-of-plane	$\mathbf{f}_{\mathbf{b}\mathbf{y}}$	=	$M_y/Z_y$			
bending - ·		=	6.94	MPa	compression	
Tension	_					
<b>3.4.3</b> Tension in rectangular tube		_	104.50	MPa		
	φFL	= OR	104.50	IVIFA		
	φFL	=	129.20	MPa		
	ΨΓL	-	125.20	IVIFA		
COMPRESSION						
3.4.8 Compression in columns, a	xial, gross	sectio	n			
1. General						3.4.8.1
Unsupported length of member	L	=	2800	mm		
Effective length factor Radius of gyration about	k	=	1.00			
buckling axis (Y)	$\mathbf{r}_{y}$	=	34.94	mm		
Radius of gyration about	r <sub>x</sub>	=	45.63	mm		
buckling axis (X)						
Slenderness ratio Slenderness ratio	kLb/ry kL/rx	=	62.97 61.36			
Cichaemess ratio	KL/1X	_	01.50			
Slenderness parameter	λ	=	0.795			
	$D_c^*$	=	39.0			
	S <sub>1</sub> *	=	0.24			
	$S_2^*$	=	1.25			
	фсс	=	0.833			
Factored limit state stress	φF∟	=	73.54	MPa		
2. Sections not subject to torsiona	al or torsio	nal-fle	vural hucklini	~		3.4.8.2
Largest slenderness ratio for				ð		0.4.0.2
flexural buckling	kL/r	=	62.97			
3.4.10 Uniform compression in co	omponents	of col	umns, gross	section -		
flat plates  1. Uniform compression in compo	nents of a	olumn	s arnss sent	ion - flat		
plates with both edges supported		Julill	o, gross sect	ivii - iial		3.4.10.1
,,,	$\mathbf{k}_1$	=	0.35			T3.3(D)
Max. distance between toes of						
fillets of supporting elements for plate	b'	=	79			
	t	=	3	mm		
Slenderness	b/t	=	26.333333			

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							_
Limit 1	S <sub>1</sub>	=	12.06				
Limit 2	S <sub>2</sub>	=	49.94				
Factored limit state stress	фГ∟	=	93.08	MPa			
Most adverse compressive limit state stress	Fa	=	73.54	MPa			
Most adverse tensile limit state stress	Fa	=	104.50	MPa			
Most adverse compressive & Tensile capacity factor	f <sub>a</sub> /F <sub>a</sub>	=	0.01		PASS		
BENDING - IN-PLANE							
<b>3.4.15</b> Compression in beams, extubes, box sections	ktreme fibro	e, gros	ss section rec	tangular			
Unbraced length for bending	L <sub>b</sub>	=	2200	mm			
Second moment of area (weak axis)	$I_y$	=	1.46E+06	mm <sup>4</sup>			
Torsion modulus	J	=	2.78E+06	$mm^3$			
Elastic section modulus	Z	=	41441.7	$mm^3$			
Slenderness	S	=	90.67				
Limit 1	$S_1$	=	21.80				
Limit 2	$S_2$	=	3854.05				
Factored limit state stress	фГ∟	=	95.00	MPa		3.4.15(2)	
<b>3.4.17</b> Compression in componer compression), gross section - flat							
nat	k <sub>1</sub>	=	0.5			T3.3(D)	
	k <sub>2</sub>	=	2.04			T3.3(D)	
Max. distance between toes of fillets of supporting elements	b'	=	79	mm		10.0(D)	
for plate							
	t	=	3	mm			
Slenderness	b/t	=	26.333333				
Limit 1	S <sub>1</sub>	=	12.06				
Limit 2	S <sub>2</sub>	=	71.35				
Factored limit state stress	фГ∟	=	93.08	MPa			
Most adverse in-plane bending limit state stress	F <sub>bx</sub>	=	93.08	MPa			
Most adverse in-plane bending capacity factor	f <sub>bx</sub> /F <sub>bx</sub>	=	0.21		PASS		

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					Î	1
BENDING - OUT-OF-PLANE	ara tha aar	ma far	out of plane l	ondina		
NOTE: Limit state stresses, $\phi F_L$ (doubly symmetric section)	are tne sar	ne tor	out-ot-plane i	penaing		
(dodbiy symmetric section)						
Factored limit state stress	φFL	=	93.08	MPa		
r dotorod imin otato otrogo	Ψ. Γ		00.00	4		
Most adverse out-of-plane						
bending limit state stress	$F_{by}$	=	93.08	MPa		
Most adverse out-of-plane			0.07		BASS	
bending capacity factor	$f_{by}/F_{by}$	=	0.07		PASS	
COMBINED ACTIONS						
4.1.1 Combined compression an	d bending					4.1.1(2)
	$F_a$	=	73.54	MPa		3.4.8
	Fao	=	93.08	MPa		3.4.10
	$F_{bx}$	=	93.08	MPa		3.4.17
	$F_by$	=	93.08	MPa		3.4.17
	i by	_	30.00	IVII G		0. 1. 17
	f <sub>a</sub> /F <sub>a</sub>	=	0.006			
						4.1.1
Check:	$f_a/F_a + f_{bx}$	/F <sub>bx</sub> + 1	$f_{\text{by}}/F_{\text{by}} \leq 1.0$			(3)
i.e.	0.29	≤	1.0		PASS	
SHEAR						
3.4.24 Shear in webs (Major						4.1.1(2)
Axis)						
Clear web height	h	_	114	mm		
Clear web neight	t	=	3	mm		
Slenderness	ر h/t	_	38	111111		
Limit 1	S <sub>1</sub>	_	33.38			
Limit 2	$S_2$	=	59.31			
Factored limit state atmosp			F7 60	MD-		
Factored limit state stress	φF∟	=	57.60	MPa		
Stress From Shear force	$f_{sx}$	=	V/A <sub>w</sub>			
2.4.25 Chaor in waha (Minar			0.00	MPa		
3.4.25 Shear in webs (Minor Axis)						
TAIO)						
Clear web height	b	=	79	mm		
	t	=	3	mm		
Slenderness	b/t	=	26.333333			
			-			
Factored limit state stress	φF∟	=	58.90	MPa		
Stress From Shear force	-	=	V/A <sub>w</sub>			
	f <sub>sy</sub>	=				

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			0.05	MPa		
Most adverseshear capacity factor (Major Axis)	$f_{sx}/F_{sx}$	=	0.00	МРа		
Most adverseshear capacity factor (Minor Axis)	$f_{sy}/F_{sy}$	=	0.00	Мра	PASS	
COMBINED ACTIONS						
	n and han	dina				
<b>4.4</b> Combined Shear, Compresion	n and bend	airig				
Check:	$f_a/F_a + f_b/F_a$	F <sub>b</sub> + (f <sub>s</sub> /	$(F_s)^2 \le 1.0$			
i.e.	0.22	≤	1.0		PASS	

### 11.1.2 Cantilever Beam



**Job no.** 21-174-2 **Date**: 17/01/2022

NAME	SYMBOL		VALUE	UNIT	NOTES	REF
60x35x3.5	Cantilever Beam					
Alloy and temper	6063-T5					AS1664.1
	F <sub>tu</sub>	=	152	MPa	Ultimate	T3.3(A)
Tension	F <sub>ty</sub>	=	110	MPa	Yield	
Compression	F <sub>cy</sub>	=	110	MPa		
Ohana	Fsu	=	90	MPa	Ultimate	
Shear	$F_{sy}$	=	62	MPa	Yield	
Pooring	$F_bu$	=	317	MPa	Ultimate	
Bearing	$F_{by}$	=	179	MPa	Yield	
Modulus of elasticity	E	=	70000	MPa	Compressiv e	
	$k_{t}$	=	1			
	<b>k</b> c	=	1			T3.4(B)
FEM ANALYSIS RESULTS						
Axial force	Р	=	0.057	kN	compressio n	
	Р	=	0	kN	Tension	

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In plane moment	M <sub>x</sub>	=	0.3859	kNm		
Out of plane moment	$M_{y}$	=	0.1841	kNm		
DESIGN STRESSES						
Gross cross section area	Ag	=	616	mm²		
In-plane elastic section modulus	$Z_{x}$	=	9420.677 8	mm³		
Out-of-plane elastic section mod.	$Z_{y}$	=	6709.733 3	mm³		
Stress from axial force	fa	=	P/A <sub>g</sub>			
		=	0.09	MPa	compressio n	
		=	0.00	MPa	Tension	
Stress from in-plane bending	$f_{bx}$	=	$M_x/Z_x$		_	
		=	40.96	MPa	compressio n	
Stress from out-of-plane	$\mathbf{f}_{by}$	=	$M_y/Z_y$		_	
bending		=	27.44	MPa	compressio n	
Tension						
3.4.3 Tension in rectangular tube	es					
	φF∟	=	104.50	MPa		
	T . L	_	10-1100	IVII a		
	4	0	104100	WII a		
		O R				
	φF <sub>L</sub>	0	129.20	МРа		
COMPRESSION		O R				
COMPRESSION 3.4.8 Compression in columns, a	фF∟	O R =				
	фF∟	O R =				3.4.8.1
<ul><li>3.4.8 Compression in columns, a</li><li>1. General</li><li>Unsupported length of</li></ul>	<b>φF</b> ∟ axial, gross sec	O R =	129.20	MPa		3.4.8.1
<ul><li>3.4.8 Compression in columns, a</li><li>1. General</li><li>Unsupported length of member</li></ul>	<b>φF</b> L axial, gross sed L	O R =	<b>129.20</b> 1850			3.4.8.1
<ul><li>3.4.8 Compression in columns, a</li><li>1. General</li><li>Unsupported length of member</li><li>Effective length factor</li></ul>	<b>φF</b> ∟ axial, gross sec	O R =	1 <b>29.20</b> 1850 1.00	MPa		3.4.8.1
<ul><li>3.4.8 Compression in columns, a</li><li>1. General</li><li>Unsupported length of member</li></ul>	<b>φF</b> L axial, gross sed L	O R =	<b>129.20</b> 1850	MPa		3.4.8.1
3.4.8 Compression in columns, a 1. General  Unsupported length of member Effective length factor Radius of gyration about buckling axis (Y) Radius of gyration about	<b>φF</b> L axial, gross sed L k	O R = ection	1 <b>29.20</b> 1850 1.00	<b>MPa</b> mm		3.4.8.1
3.4.8 Compression in columns, a 1. General  Unsupported length of member Effective length factor Radius of gyration about buckling axis (Y)	φF <sub>L</sub> axial, gross sec  L  k  r <sub>y</sub>	O R = ection	1850 1.00 13.81	MPa mm		3.4.8.1
3.4.8 Compression in columns, a 1. General  Unsupported length of member Effective length factor Radius of gyration about buckling axis (Y) Radius of gyration about buckling axis (X)	φF <sub>L</sub> axial, gross sec  L  k  r <sub>y</sub> r <sub>x</sub>	O R = ction	1850 1.00 13.81 21.42	MPa mm		3.4.8.1
3.4.8 Compression in columns, a 1. General  Unsupported length of member Effective length factor Radius of gyration about buckling axis (Y) Radius of gyration about buckling axis (X) Slenderness ratio	φF <sub>L</sub> axial, gross sec  L  k  r <sub>y</sub> r <sub>x</sub> kLb/ry	O R =	1850 1.00 13.81 21.42 68.81	MPa mm		3.4.8.1
3.4.8 Compression in columns, a 1. General  Unsupported length of member Effective length factor Radius of gyration about buckling axis (Y) Radius of gyration about buckling axis (X) Slenderness ratio Slenderness ratio	φF <sub>L</sub> axial, gross sec  k  r <sub>y</sub> r <sub>x</sub> kLb/ry  kL/rx	O R =	1850 1.00 13.81 21.42 68.81 86.37	MPa mm		3.4.8.1
3.4.8 Compression in columns, a 1. General  Unsupported length of member Effective length factor Radius of gyration about buckling axis (Y) Radius of gyration about buckling axis (X) Slenderness ratio Slenderness ratio	φF <sub>L</sub> axial, gross sec  L  k  r <sub>y</sub> r <sub>x</sub> kLb/ry  kL/rx	O R = = = = = = = = = = = = = = = = = =	1850 1.00 13.81 21.42 68.81 86.37 1.09	MPa mm		3.4.8.1
3.4.8 Compression in columns, a 1. General  Unsupported length of member Effective length factor Radius of gyration about buckling axis (Y) Radius of gyration about buckling axis (X) Slenderness ratio Slenderness ratio	φF <sub>L</sub> axial, gross sec  L  k  r <sub>y</sub> r <sub>x</sub> kLb/ry  kL/rx  λ  D <sub>c</sub> *	O R = = = = = = = = = = = = = = = = = =	1850 1.00 13.81 21.42 68.81 86.37 1.09 39.0	MPa mm		3.4.8.1
3.4.8 Compression in columns, a 1. General  Unsupported length of member Effective length factor Radius of gyration about buckling axis (Y) Radius of gyration about buckling axis (X) Slenderness ratio Slenderness ratio	φF <sub>L</sub> axial, gross sec  L  k  ry  r <sub>x</sub> kLb/ry  kL/rx  λ  D <sub>c</sub> *  S <sub>1</sub> *	O R =	1850 1.00 13.81 21.42 68.81 86.37 1.09 39.0 0.24	MPa mm		3.4.8.1

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	$k_2$		2.04		1	T3.3(D)
<b>3.4.17</b> Compression in componer compression), gross section - flat						T3.3(D)
Factored limit state stress	фF∟	=	94.54	МРа		3.4.15(2)
Limit 2	S <sub>2</sub>	=	3854.05			
Limit 1	S <sub>1</sub>	=	21.80			
Slenderness	S	=	104.06			
Elastic section modulus	Z	=	8	mm³		
Torsion modulus	J	=	2.52E+05 9420.677	mm <sup>3</sup>		
(weak axis)	l <sub>y</sub>	=	1.17E+05	mm <sup>4</sup>		
Unbraced length for bending Second moment of area	L <sub>b</sub>	=	950	mm		
BENDING - IN-PLANE <b>3.4.15</b> Compression in beams, ex tubes, box sections		gross se	ection rectang	gular		
Most adverse compressive & Tensile capacity factor	f <sub>a</sub> /F <sub>a</sub>	=	0.00		PASS	
Most adverse tensile limit state stress  Most adverse compressive &	Fa	=	104.50	MPa		
Most adverse compressive limit state stress	Fa	=	59.18	MPa		
Factored limit state stress	φFL	=	104.50	MPa		
Limit 2	S <sub>2</sub>	=	49.94			
Limit 1	S <sub>1</sub>	=	12.06			
Slenderness	b/t	=	8			
·	t	=	3.5	mm		
Max. distance between toes of fillets of supporting elements for plate	b'	=	28			
olatee war bear eagee capported	$\mathbf{k}_1$	=	0.35			T3.3(D
<b>3.4.10</b> Uniform compression in conplates  1. Uniform compression in compoplates with both edges supported	onents of colu					3.4.10.
flexural buckling				da a Mar		
Largest slenderness ratio for	kL/r	=	86.37			

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Max. distance between toes of fillets of supporting	b'	=	28	mm		
elements for plate						
Slenderness	t b/t	=	3.5 8	mm		
Limit 1	S <sub>1</sub>	=	12.06			
Limit 2	$S_2$	=	71.35			
Factored limit state stress	фҒ∟	=	104.50	МРа		
Most adverse in-plane bending limit state stress	F <sub>bx</sub>	=	94.54	MPa		
Most adverse in-plane bending capacity factor	f <sub>bx</sub> /F <sub>bx</sub>	=	0.43		PASS	
BENDING - OUT-OF-PLANE	aro the same	for out o	f nlana har	dina		
NOTE: Limit state stresses, φF <sub>L</sub> a (doubly symmetric section)	are trie same i	or out-o	-piarie ben	uing		
Factored limit state stress	фҒ∟	=	94.54	MPa		
Most adverse out-of-plane bending limit state stress	F <sub>by</sub>	=	94.54	MPa		
Most adverse out-of-plane bending capacity factor	$f_{by}/F_{by}$	=	0.29		PASS	
COMBINED ACTIONS						
4.1.1 Combined compression and	d bending					4.1.1(2)
	Fa	=	59.18	MPa		3.4.8
	$F_{ao}$	=	104.50	MPa		3.4.10
	$F_{bx}$	=	94.54	MPa		3.4.17
	$F_{by}$	=	94.54	MPa		3.4.17
	f <sub>a</sub> /F <sub>a</sub>	=	0.002			
Check:	$f_a/F_a + f_{bx}/F_{bx} +$	f <sub>by</sub> /F <sub>by</sub> :	≤ 1.0			4.1.1
i.e.	0.73	≤	1.0		PASS	
SHEAR						
<b>3.4.24</b> Shear in webs (Major Axis)						4.1.1(2)

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			45 44005		1	ı
Slenderness	h/t	=	15.14285 7			
Limit 1	S <sub>1</sub>	=	33.38			
Limit 2	$S_2$	=	59.31			
Factored limit state stress	фҒ∟	=	58.90	MPa		
Stress From Shear force	$f_{sx}$	=	V/A <sub>w</sub>			
			0.93	MPa		
<b>3.4.25</b> Shear in webs (Minor Axis)						
-,						
Clear web height	b	=	28	mm		
	t	=	3.5	mm		
Slenderness	b/t	=	8			
Factored limit state stress	φF∟	=	58.90	MPa		
Stress From Shear force	f <sub>sy</sub>	=	$V/A_w$			
	·		0.64	MPa		
Most adverseshear capacity					$\dashv$	
factor (Major Axis)	$f_{sx}/F_{sx}$	=	0.02	MPa		
Most adverseshear capacity factor (Minor Axis)	$f_{sy}/F_{sy}$	=	0.01	Мра	PASS	
· · · · · · · · · · · · · · · · · · ·						
COMBINED ACTIONS						
4.4 Combined Shear, Compresion	n and bending	9				
_						
Check:	$f_a/F_a + f_b/F_b + f_a$	$(f_s/F_s)^2 \le$	1.0			

## 11.1.3 Brace (typ.1)



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**Job no.** 21-174-2 **Date**: 17/01/2022

NAME	SYMBOL		VALUE	UNIT	NOTES	REF
60x35x3.5	Brace 1					
Alloy and temper	6063-T5					AS1664.1
Tanaian	$F_{tu}$	=	152	MPa	Ultimate	T3.3(A)
Tension	$F_{ty}$	=	110	MPa	Yield	
Compression	F <sub>cy</sub>	=	110	MPa		

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	$F_su$	=	90	MPa	Ultimate	
Shear	$F_{sy}$	=	62	MPa	Yield	
	$F_{bu}$	=	317	MPa	Ultimate	
Bearing	F <sub>by</sub>	=	179	MPa	Yield	
	. 5,			🍝	170.0	
Modulus of elasticity	Е	=	70000	MPa	Compressive	
	<b>k</b> t	=	1			T3.4(B)
	<b>k</b> c	=	1			10.1(2)
FEM ANALYSIS RESULTS						
Axial force	Р	=	0.117	kN	compression	
	Р	=	0	kN	Tension	
In plane moment	$M_{x}$	=	7.227E-19	kNm		
Out of plane moment	$M_{y}$	=	0.2362	kNm		
DEGICAL OTREGOES						
DESIGN STRESSES Gross cross section area	۸		616	mm <sup>2</sup>		
In-plane elastic section	$A_g$	=				
modulus	$Z_{x}$	=	9420.6778	mm³		
Out-of-plane elastic section mod.	$Z_{y}$	=	6709.7333	mm³		
Stress from axial force	fa	=	P/A <sub>g</sub>			
		=	0.19	MPa	compression	
		=	0.00	MPa	Tension	
Stress from in-plane bending	$f_{bx}$	=	$M_x/Z_x$			
	_	=	0.00	MPa	compression	
Stress from out-of-plane	$f_{by}$	=	$M_y/Z_y$			
bending 		=	35.20	MPa	compression	
Tension						
3.4.3 Tension in rectangular tubes			404.50	MD.		
	фҒ∟	=	104.50	MPa		
	<b>4</b> E	OR	120.20	MDo		
	фҒ∟	=	129.20	MPa		
COMPRESSION						
<b>3.4.8</b> Compression in columns, ax 1. General	cial, gross	section	1			3.4.8.1
Unsupported length of member	L	=	1000	mm		
Effective length factor	k	=	1.00	111111		
Radius of gyration about						
buckling axis (Y)	r <sub>y</sub>	=	13.81	mm		

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Radius of gyration about buckling axis (X)	r <sub>x</sub>	=	21.42	mm		
Slenderness ratio	kLb/ry	=	72.43			
Slenderness ratio	kL/rx	=	46.69			
Slenderness parameter	λ	=	0.91			
	$D_c^*$	=	39.0			
	$S_1^*$	=	0.24			
	$S_2^*$	=	1.25			
	фсс	=	0.808			
Factored limit state stress	фГ∟	=	67.56	MPa		
2. Sections not subject to torsiona	al or torsior	nal-flex	kural buckling	1		3.4.8.2
Largest slenderness ratio for flexural buckling	kL/r	=	72.43			
<b>3.4.10</b> Uniform compression in coflat plates	mponents	of colu	umns, gross s	section -		
1. Uniform compression in components of columns, gross section - flat plates with both edges supported						3.4.10.1
	<b>k</b> <sub>1</sub>	=	0.35			T3.3(D)
Max. distance between toes of fillets of supporting elements for plate	b'	=	28			
, p	t	=	3.5	mm		
Slenderness	b/t	=	8			
Limit 1	$S_1$	=	12.06			
Limit 2	$S_2$	=	49.94			
Factored limit state stress	фГ∟	=	104.50	MPa		
Most adverse compressive limit	Fa	_	67.56	MPa	1	
state stress	га	=	07.50	IVIFA		
Most adverse tensile limit state stress	Fa	=	104.50	MPa		
Most adverse compressive & Tensile capacity factor	f <sub>a</sub> /F <sub>a</sub>	=	0.00		PASS	
BENDING - IN-PLANE						
<b>3.4.15</b> Compression in beams, extubes, box sections	treme fibre	e, gros	s section rec	tangular		
Unbraced length for bending	L <sub>b</sub>	=	1000	mm		
Second moment of area (weak axis)	l <sub>y</sub>	=	117420.33	mm <sup>4</sup>		

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Torsion modulus	J	=	251961.03	mm³		
Elastic section modulus	Z	=	9420.6778	${\sf mm}^3$		
Slenderness	S	=	109.54			
Limit 1	S <sub>1</sub>	=	21.80			
Limit 2	$S_2$	=	3854.05			
Factored limit state stress	φF <sub>L</sub>	=	94.37	MPa		3.4.15(2)
3.4.17 Compression in componer compression), gross section - flat						
, ,, ,						To o(D)
	<b>k</b> <sub>1</sub>	=	0.5			T3.3(D)
	<b>k</b> <sub>2</sub>	=	2.04			T3.3(D)
Max. distance between toes of			20			
fillets of supporting elements for plate	b'	=	28	mm		
ioi piate	t	_	3.5	mm		
Slenderness	b/t	=	3.5 8	111111		
Limit 1	S <sub>1</sub>	=	12.06			
Limit 2	S <sub>2</sub>	=	71.35			
LIIIII Z	32	=	71.33			
Factored limit state stress	φF∟	=	104.50	MPa		
Most adverse in-plane bending limit state stress	F <sub>bx</sub>	=	94.37	MPa		
Most adverse in-plane bending capacity factor	f <sub>bx</sub> /F <sub>bx</sub>	=	0.00		PASS	
BENDING - OUT-OF-PLANE						
	46	fo	t of mlone b	a 10 alim a		
NOTE: Limit state stresses, φF <sub>L</sub> a (doubly symmetric section)	are tne sarr	ie ior (	out-ot-plane b	enaing		
Factored limit state stress	фҒ∟	=	94.37	MPa		
Most adverse out-of-plane	F <sub>by</sub>	=	94.37	MPa	1	
bending limit state stress	- Dy		5 .101	🏎		
Most adverse out-of-plane bending capacity factor	f <sub>by</sub> /F <sub>by</sub>	=	0.37		PASS	
COMBINED ACTIONS						
4.1.1 Combined compression and	d bending					4.1.1(2)
	Fa	=	67.56	MPa		3.4.8
	F <sub>ao</sub>	=	104.50	MPa		3.4.10
	ı ao	_	104.50	IVIFa	1	5.4.10

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	$F_bx$	=	94.37	MPa		3.4.17
	$F_{by}$	=	94.37	MPa		3.4.17
	$f_a/F_a$	=	0.003			
Check:	$f_a/F_a + f_{bx}$	/F <sub>bx</sub> + f	$_{by}/F_{by} \le 1.0$			4.1.1
i.e.	0.38	≤	1.0		PASS	(0)
SHEAR						
3.4.24 Shear in webs (Major						4.1.1(2)
Axis)						,
Clear web height	h	=	53	mm		
	t	=	3.5	mm		
Slenderness	h/t	=	15.142857			
Limit 1	S <sub>1</sub>	=	33.38			
Limit 2	$S_2$	=	59.31			
Littiit Z	02	_	39.31			
Factored limit state stress	φFL	=	58.90	MPa		
Stress From Shear force	f <sub>sx</sub>	=	V/A <sub>w</sub>			
	- CA		0.02	MPa		
3.4.25 Shear in webs (Minor						
Axis)						
Clear web beight	b	_	28	mm		
Clear web height	t	=	3.5	mm mm		
Slenderness	b/t	=	8	111111		
Oleride Mess	D/ t	-	O			
Factored limit state stress	φFL	=	58.90	MPa		
Stress From Shear force	f <sub>sy</sub>	=	V/A <sub>w</sub>			
	-,		0.73	MPa		
Most adverseshear capacity	f <sub>sx</sub> /F <sub>sx</sub>	_	0.00	MPa	1	
factor (Major Axis)	ISX/ FSX	=	0.00	IVIFA		
Most adverseshear capacity	$f_{sy}/F_{sy}$	=	0.01	Мра	PASS	
factor (Minor Axis)	· sy/ · sy		0.01		. , .00	
COMPINED ACTIONS						
COMBINED ACTIONS	n and han	dina				
<b>4.4</b> Combined Shear, Compresion	ni and bene	uirig				
Check:	f_/F, f. /l	F. + /f	$(F_{s})^{2} \le 1.0$			
					DACC	
i.e.	0.38	≤	1.0		PASS	l



# 11.1.4 Brace (typ.2)



**Job no.** 21-174-2 **Date:** 17/01/2022

NAME	SYMBOL		VALUE	UNIT	NOTES	REF
30x20x1.5	Brace 2					
Alloy and temper	6063-T5					AS1664.1
Tension	$F_{tu}$	=	152	MPa	Ultimate	T3.3(A)
Tension	$F_{ty}$	=	110	MPa	Yield	
Compression	$F_{cy}$	=	110	MPa		
Shear	$F_{su}$	=	90	MPa	Ultimate	
Oriear	$F_{sy}$	=	62	MPa	Yield	
Pooring	$F_bu$	=	317	MPa	Ultimate	
Bearing	$F_by$	=	179	MPa	Yield	
Modulus of elasticity	Е	=	70000	MPa	Compressive	
	$k_{t}$	=	1			T2 4/D)
	<b>k</b> c	=	1			T3.4(B)
FEM ANALYSIS RESULTS						
Axial force	Р	=	0.272	kN	compression	
	Р	=	0	kN	Tension	
In plane moment	$M_{x}$	=	0	kNm		
Out of plane moment	$M_y$	=	0.0154	kNm		
DESIGN STRESSES						
Gross cross section area	$A_g$	=	141	mm²		
In-plane elastic section modulus	$Z_{x}$	=	1141.05	mm³		
Out-of-plane elastic section mod.	$Z_{y}$	=	894.575	mm³		
Stress from axial force	fa	=	P/A <sub>g</sub>			
		=	1.93	MPa	compression	
	_	=	0.00	MPa	Tension	
Stress from in-plane bending	$f_{bx}$	=	$M_x/Z_x$	MD.		
		=	0.00	MPa	compression	
	$\mathbf{f}_{by}$	=	$M_y/Z_y$			

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Stress from out-of-plane bending		=	17.21	MPa	compression	
Tension						
3.4.3 Tension in rectangular tube	S					
	φF <sub>L</sub>	= OR	104.50	MPa		
	φFL	=	129.20	MPa		
COMPRESSION						
<b>3.4.8</b> Compression in columns, at 1. General	xial, gross	section	1			3.4.8.1
Unsupported length of member	L	=	1000	mm		
Effective length factor	k	=	1.00			
Radius of gyration about buckling axis (Y)	$\mathbf{r}_{y}$	=	7.97	mm		
Radius of gyration about buckling axis (X)	$r_{x}$	=	11.02	mm		
Slenderness ratio	kLb/ry	=	125.55			
Slenderness ratio	kL/rx	=	90.76			
Slenderness parameter	λ	=	1.58			
	D <sub>c</sub> *	=	39.0			
	S <sub>1</sub> *	=	0.24			
	S <sub>2</sub> *	=	1.25			
	фсс	=	0.802			
Factored limit state stress	φF <sub>L</sub>	=	35.14	MPa		
2. Sections not subject to torsiona	al or torsio	nal-flex	ural buckling	1		3.4.8.2
Largest slenderness ratio for flexural buckling	kL/r	=	125.55			
3.4.10 Uniform compression in coflat plates	omponents	of colu	ımns, gross s	section -		
Uniform compression in compo plates with both edges supported		olumns	, gross secti	on - flat		3.4.10.1
	<b>k</b> <sub>1</sub>	=	0.35			T3.3(D)
Max. distance between toes of fillets of supporting elements for plate	b'	=	17			
1	t	=	1.5	mm		
Slenderness	b/t	=	11.333333			
Limit 1	S <sub>1</sub>	=	12.06			
Limit 2	$S_2$	=	49.94			
Lillin Z	<b>J</b> <sub>2</sub>	_	70.07		I	I

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	фГ∟	=	104.50	MPa		
Most adverse compressive limit state stress	Fa	=	35.14	MPa		
Most adverse tensile limit state stress	Fa	=	104.50	MPa		
Most adverse compressive & Tensile capacity factor	f <sub>a</sub> /F <sub>a</sub>	=	0.05		PASS	
BENDING - IN-PLANE						
<b>3.4.15</b> Compression in beams, ex tubes, box sections	treme fibre	e, gros	s section rect	angular		
Unbraced length for bending	L <sub>b</sub>	=	1000	mm		
Second moment of area (weak axis)	ly	=	8945.75	mm <sup>4</sup>		
Torsion modulus	J	=	17744.206	mm³		
Elastic section modulus	Z	=	1141.05	mm³		
Slenderness	S	=	181.13			
Limit 1	S <sub>1</sub>	=	21.80			
Limit 2	S <sub>2</sub>	=	3854.05			
Factored limit state stress	фF∟	=	92.36	MPa		3.4.15(i
3.4.17 Compression in component compression), gross section - flat						
compression, gross section hat	prace ma	i botti	cages suppo	100		
compression), gross section mat		=		ica		T3.3(I
compression, gross seedien mat	k <sub>1</sub>	=	0.5	icu		
·				iou		
Max. distance between toes of fillets of supporting elements	k <sub>1</sub>	=	0.5	mm		
Max. distance between toes of	k <sub>1</sub> k <sub>2</sub>	=	0.5 2.04			T3.3(I
Max. distance between toes of fillets of supporting elements	k <sub>1</sub> k <sub>2</sub> b'	= =	0.5 2.04	mm		
Max. distance between toes of fillets of supporting elements for plate	k <sub>1</sub> k <sub>2</sub> b' t	= =	0.5 2.04 17 1.5	mm		
Max. distance between toes of fillets of supporting elements for plate  Slenderness	k <sub>1</sub> k <sub>2</sub> b' t b/t	= = = =	0.5 2.04 17 1.5 11.333333	mm		
Max. distance between toes of fillets of supporting elements for plate  Slenderness Limit 1	k <sub>1</sub> k <sub>2</sub> b' t b/t S <sub>1</sub>	= = = = =	0.5 2.04 17 1.5 11.333333 12.06	mm		
Max. distance between toes of fillets of supporting elements for plate  Slenderness Limit 1 Limit 2	k <sub>1</sub> k <sub>2</sub> b' t b/t S <sub>1</sub> S <sub>2</sub>	= = = = =	0.5 2.04 17 1.5 11.333333 12.06 71.35	mm mm		

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BENDING - OUT-OF-PLANE						
NOTE: Limit state stresses, $\phi F_L$ (doubly symmetric section)	are the san	ne for d	out-of-plane b	ending		
Factored limit state stress	фҒ∟	=	92.36	MPa		
Most adverse out-of-plane bending limit state stress	F <sub>by</sub>	=	92.36	MPa		
Most adverse out-of-plane bending capacity factor	f <sub>by</sub> /F <sub>by</sub>	=	0.19		PASS	
COMBINED ACTIONS						
4.1.1 Combined compression ar	nd bending					4.1.1(2)
	Fa	=	35.14	MPa		3.4.8
	Fao		104.50	MPa		3.4.10
	$F_{bx}$	=	92.36	MPa		3.4.17
	$F_{by}$	=	92.36	MPa		3.4.17
	f <sub>a</sub> /F <sub>a</sub>	=	0.055			
Check:	$f_a/F_a + f_{bx}/$	F <sub>bx</sub> + f	$_{by}/F_{by} \le 1.0$			4.1.1
i.e.	0.24	≤	1.0		PASS	
SHEAR						
<b>3.4.24</b> Shear in webs (Major Axis)						4.1.1(2)
Clear web height	h	=	27	mm		
<b>-</b>	t	=	1.5	mm		
Slenderness Limit 1	h/t S₁	=	18 33.38			
Limit 2	S <sub>1</sub>	=	59.31			
LIIIII Z	32	=	39.31			
Factored limit state stress	φF∟	=	58.90	MPa		
Stress From Shear force	$\mathbf{f}_{\mathbf{sx}}$	=	$V/A_w$			
3.4.25 Shear in webs (Minor Axis)			0.01	MPa		
Clear web height	b	=	17	mm		
Slenderness	t b/t	=	1.5 11.333333	mm		
				MDo		
Factored limit state stress Stress From Shear force	φF∟ f	=	<b>58.90</b> V/A <sub>w</sub>	MPa		
Oness i form offeat force	f <sub>sy</sub>	=	v/~w		l	<u> </u>

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			0.22	MPa	
Most adverseshear capacity factor (Major Axis)	f <sub>sx</sub> /F <sub>sx</sub>	=	0.00	МРа	
Most adverseshear capacity factor (Minor Axis)	$f_{sy}/F_{sy}$	=	0.00	Мра	PASS
COMBINED ACTIONS	on and hand	lina			
<b>4.4</b> Combined Shear, Compresion	f <sub>a</sub> /F <sub>a</sub> + f <sub>b</sub> /F	J	$F_0^2 < 1.0$		
i.e.		≤ (1s/1	1.0		PASS

# 11.1.5 Middle Beam



**Job no.** 21-174-2 **Date**: 17/01/2022

NAME	SYMBOL		VALUE	UNIT	NOTES	REF
30x20x1.5	Middle Beam					
Alloy and temper	6063-T5					AS1664.1
	F <sub>tu</sub>	=	152	MPa	Ultimate	T3.3(A)
Tension	F <sub>ty</sub>	=	110	MPa	Yield	( )
Compression	F <sub>cy</sub>	=	110	MPa		
Chaor	$F_su$	=	90	MPa	Ultimate	
Shear	$F_{sy}$	=	62	MPa	Yield	
Bearing	$F_bu$	=	317	MPa	Ultimate	
Dearing	$F_by$	=	179	MPa	Yield	
Modulus of elasticity	E	=	70000	MPa	Compressive	
	$\mathbf{k}_{t}$	=	1			TO 4(D)
	<b>k</b> <sub>c</sub>	=	1			T3.4(B)
FEM ANALYSIS RESULTS						
	_		_			
Axial force	P	=	0	kN	compression	
	P	=	0.162	kN	Tension	
In plane moment	$M_x$	=	0.0477	kNm		

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Out of plane moment	$M_{y}$	=	0.0018	kNm		
DESIGN STRESSES						
Gross cross section area	Ag	=	141	mm²		
In-plane elastic section	$Z_{x}$	=	1141.05	mm³		
modulus	<b>-</b> x	_	1141.00			
Out-of-plane elastic section mod.	$Z_y$	=	894.575	mm³		
Stress from axial force	fa	=	P/A <sub>g</sub>			
		=	0.00	MPa	compression	
		=	1.15	MPa	Tension	
Stress from in-plane bending	$f_{bx}$	=	$M_x/Z_x$			
	_	=	41.80	MPa	compression	
Stress from out-of-plane	$f_{by}$	=	$M_y/Z_y$			
bending 		=	2.01	MPa	compression	
Tension						
3.4.3 Tension in rectangular tubes			404.50	MD.		
	φF∟	=	104.50	MPa		
	4.5	OR	400.00	MDa		
	φF∟	=	129.20	MPa		
COMPRESSION						
3.4.8 Compression in columns, as	kial, gross se	ection				
1. General						3.4.8.1
Unsupported length of member	L	=	2040	mm		
Effective length factor	k	=	1.00			
Radius of gyration about	r <sub>y</sub>	=	7.97	mm		
buckling axis (Y)	Ty	_	1.31	111111		
Radius of gyration about buckling axis (X)	r <sub>x</sub>	=	11.02	mm		
Slenderness ratio	kLb/ry	=	130.57			
Slenderness ratio	kL/rx	=	185.16			
Slenderness parameter	λ	=	2.34			
Oleridemess parameter	D <sub>c</sub> *	=	39.0			
	Տ₁*	=	0.24			
	S <sub>2</sub> *		1.25			
		=				
	фсс	=	0.907			
Factored limit state stress	φFL	=	18.28	MPa		
2. Sections not subject to torsiona	al or torsiona	al-flexura	al buckling			3.4.8.2
Largest slenderness ratio for	kL/r	=	185.16			
flexural buckling	<del>, -</del>					

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1. Uniform compression in compo	nents of co	lumns,	gross section	- flat		3.4.10.1
plates with both edges supported	le.		0.25			
May distance between tops of	<b>k</b> 1	=	0.35			T3.3(D)
Max. distance between toes of fillets of supporting elements for plate	b'	=	17			
•	t	=	1.5	mm		
Slenderness	b/t	=	11.333333			
Limit 1	S <sub>1</sub>	=	12.06			
Limit 2	$S_2$	=	49.94			
Factored limit state stress	φFL	=	104.50	MPa		
Most adverse compressive limit state stress	Fa	=	18.28	MPa		ļ 
Most adverse tensile limit state stress	Fa	=	104.50	MPa		
Most adverse compressive & Tensile capacity factor	f <sub>a</sub> /F <sub>a</sub>	=	0.01		PASS	
BENDING - IN-PLANE  3.4.15 Compression in beams, exi	treme fibre,	gross	section rectar	ngular		
BENDING - IN-PLANE  3.4.15 Compression in beams, exitubes, box sections	treme fibre, L <sub>b</sub>	gross =	section rectar 1040	ngular mm		
BENDING - IN-PLANE  3.4.15 Compression in beams, extubes, box sections  Unbraced length for bending Second moment of area (weak		=				
BENDING - IN-PLANE  3.4.15 Compression in beams, exitubes, box sections  Unbraced length for bending Second moment of area (weak axis)	L <sub>b</sub>	=	1040	mm mm <sup>4</sup>		
BENDING - IN-PLANE  3.4.15 Compression in beams, extubes, box sections  Unbraced length for bending Second moment of area (weak axis)  Torsion modulus	$L_b$	=	1040 8945.75	mm		
BENDING - IN-PLANE  3.4.15 Compression in beams, extubes, box sections  Unbraced length for bending Second moment of area (weak axis)  Torsion modulus Elastic section modulus	L <sub>b</sub> I <sub>y</sub> J	=	1040 8945.75 17744.206	mm mm <sup>4</sup> mm <sup>3</sup>		
BENDING - IN-PLANE  3.4.15 Compression in beams, exitubes, box sections  Unbraced length for bending Second moment of area (weak axis) Torsion modulus Elastic section modulus Slenderness	L <sub>b</sub> Iy J Z	= = =	1040 8945.75 17744.206 1141.05	mm mm <sup>4</sup> mm <sup>3</sup>		
BENDING - IN-PLANE  3.4.15 Compression in beams, exitubes, box sections  Unbraced length for bending Second moment of area (weak axis)  Torsion modulus  Elastic section modulus  Slenderness  Limit 1	L <sub>b</sub> I <sub>y</sub> J Z S	= = =	1040 8945.75 17744.206 1141.05 188.38	mm mm <sup>4</sup> mm <sup>3</sup>		
BENDING - IN-PLANE  3.4.15 Compression in beams, exitubes, box sections  Unbraced length for bending Second moment of area (weak axis)  Torsion modulus Elastic section modulus Slenderness Limit 1 Limit 2	L <sub>b</sub> I <sub>y</sub> J Z S S1	= = = =	1040 8945.75 17744.206 1141.05 188.38 21.80	mm mm <sup>4</sup> mm <sup>3</sup>		 3.4.15(2
BENDING - IN-PLANE  3.4.15 Compression in beams, exitubes, box sections  Unbraced length for bending Second moment of area (weak axis) Torsion modulus Elastic section modulus Slenderness Limit 1 Limit 2  Factored limit state stress	L <sub>b</sub> I <sub>y</sub> J Z S S <sub>1</sub> S <sub>2</sub> <b>ΦF</b> L	= = = = = = =	1040 8945.75 17744.206 1141.05 188.38 21.80 3854.05 <b>92.19</b>	mm  mm <sup>4</sup> mm <sup>3</sup> mm <sup>3</sup>		3.4.15(2
BENDING - IN-PLANE	L <sub>b</sub> I <sub>y</sub> J Z S S <sub>1</sub> S <sub>2</sub> <b>ΦF</b> L	= = = = = = =	1040 8945.75 17744.206 1141.05 188.38 21.80 3854.05 <b>92.19</b>	mm  mm <sup>4</sup> mm <sup>3</sup> mm <sup>3</sup>		3.4.15(2 T3.3(D

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Max. distance between toes of fillets of supporting elements for plate	b'	=	17	mm		
	t	=	1.5	mm		
Slenderness	b/t	=	11.333333			
Limit 1	S <sub>1</sub>	=	12.06			
Limit 2	$S_2$	=	71.35			
Factored limit state stress	фF∟	=	104.50	MPa		
Most adverse in-plane bending limit state stress	F <sub>bx</sub>	=	92.19	MPa		
Most adverse in-plane bending capacity factor	f <sub>bx</sub> /F <sub>bx</sub>	=	0.45		PASS	
BENDING - OUT-OF-PLANE						
NOTE: Limit state stresses, φF <sub>L</sub> a (doubly symmetric section)	are the same	for out	t-of-plane ber	nding		
Factored limit state stress	фҒ∟	=	92.19	MPa		
Most adverse out-of-plane bending limit state stress	$F_by$	=	92.19	MPa		
Most adverse out-of-plane bending capacity factor	f <sub>by</sub> /F <sub>by</sub>	=	0.02		PASS	
COMBINED ACTIONS						
4.1.1 Combined compression and	d bending					4.1.1(2)
	Fa	=	18.28	MPa		3.4.8
	Fao	=	104.50	MPa		3.4.10
	$F_bx$	=	92.19	MPa		3.4.17
	$F_by$	=	92.19	MPa		3.4.17
	f <sub>a</sub> /F <sub>a</sub>	=	0.011			
Check:	$f_a/F_a + f_{bx}/F_b$	x + fhv/l				4.1.1
i.e.	0.49	,, · · ·by, · ≤	1.0		PASS	(3)
SHEAR 3.4.24 Shear in webs (Major Axis)						4.1.1(2)
,						
Clear web height	h	=	27	mm		
Slenderness	t h/t	=	1.5 18	mm		

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Limit 1	S <sub>1</sub>	_	33.38		1
		=			
Limit 2	$S_2$	=	59.31		
Factored limit state stress	φF∟	_	58.90	MPa	
	•	=		IVIFA	
Stress From Shear force	$f_{sx}$	=	V/A <sub>w</sub>		
0.405.01			0.59	MPa	
3.4.25 Shear in webs (Minor					
Axis)					
Clear web height	b	=	17	mm	
-	t	=	1.5	mm	
Slenderness	b/t	=	11.333333		
Factored limit state stress	φF∟	=	58.90	MPa	
Stress From Shear force	$f_{sy}$	=	$V/A_w$		
			0.10	MPa	
Most adverseshear capacity	f <sub>sx</sub> /F <sub>sx</sub>	=	0.01	MPa	
factor (Major Axis)	13% 1 3%	_	0.01	u	
Most adverseshear capacity	f <sub>sy</sub> /F <sub>sy</sub>	=	0.00	Мра	PASS
factor (Minor Axis)				•	
COMPINED ACTIONS					
COMBINED ACTIONS  4.4 Combined Shear, Compresion	n and handin	200			
4.4 Combined Shear, Compresion	ii and bendin	iy			
Check:	$f_a/F_a + f_b/F_b$	+ (f <sub>a</sub> /F <sub>a</sub>	o <sup>2</sup> < 1 ∩		
					DACC
i.e.	0.46	≤	1.0		PASS

# 11.1.6 Corner Beam



Email: info@primeengineers.com.au

**Job no.** 21-174-2 **Date**: 17/01/2022

NAME	SYMBOL		VALUE	UNIT	NOTES	REF
30x20x1.5	Corner Beam					
Alloy and temper	6063-T5					AS1664.1
Tanaian	Ftu	=	152	MPa	Ultimate	T3.3(A)
Tension	$F_{ty}$	=	110	MPa	Yield	
Compression	F <sub>cy</sub>	=	110	MPa		
Shear	F <sub>su</sub>	=	90	MPa	Ultimate	

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	$F_{sy}$	=	62	MPa	Yield	
	F <sub>bu</sub>	=	317	MPa	Ultimate	
Bearing	F <sub>by</sub>	=	179	MPa	Yield	
	-,					
Modulus of elasticity	Е	=	70000	MPa	Compressiv	
					е	
	$k_{t}$	=	1			
	k <sub>c</sub>	=	1			T3.4(B)
FEM ANALYSIS RESULTS						
Axial force	Р	=	0	kN	compression	
Axial force	' P	=	0.201	kN	Tension	
In plane moment	M <sub>x</sub>	=	0.201	kNm	TOTISION	
Out of plane moment	M <sub>y</sub>	=	0.0062	kNm		
Cut of plane memoria	···y		0.0002			
DESIGN STRESSES						
Gross cross section area	$A_g$	=	141	mm <sup>2</sup>		
In-plane elastic section modulus	$Z_{x}$	=	1141.05	mm³		
Out-of-plane elastic section	7		004 575	mm³		
mod.	$Z_{y}$	=	894.575	IIIIII		
Stress from axial force	fa	=	P/A <sub>g</sub>			
		=	0.00 1.43	MPa MPa	compression Tension	
Stress from in-plane bending	$f_{bx}$	=	1.43 M <sub>x</sub> /Z <sub>x</sub>	IVIFA	Terision	
oneds from in plane bending	• DX	=	69.06	MPa	compression	
Stress from out-of-plane	$f_{by}$	=	$M_y/Z_y$	•		
bending	,	=	6.93	MPa	compression	
Tension						
3.4.3 Tension in rectangular tubes						
	φFL	=	104.50	MPa		
		O R				
	φFL	=	129.20	MPa		
	, -					
COMPRESSION						
<b>3.4.8</b> Compression in columns, ax 1. General	ial, gross s	ection				3.4.8.1
Unsupported length of member	L	=	2820	mm		
Effective length factor	k	=	1.00	<b>-</b>		
Radius of gyration about	$\mathbf{r}_{y}$	_	7.97	mm		
buckling axis (Y)	Ту	=	1.31	111111		

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Radius of gyration about buckling axis (X)	r <sub>x</sub>	=	11.02	mm		
Slenderness ratio	kLb/ry	=	232.26			
Slenderness ratio	kL/rx	=	255.95			
	,		0.00			
Slenderness parameter	λ	=	3.23			
	D <sub>c</sub> * S₁*	=	39.0			
	S <sub>1</sub>	=	0.24			
		=	1.25			
	фсс	=	0.950			
Factored limit state stress	φF∟	=	10.02	MPa		
2. Sections not subject to torsiona	al or torsiona	ıl-flexur	al buckling			3.4.8.2
Largest slenderness ratio for flexural buckling	kL/r	=	255.95			
3.4.10 Uniform compression in co	mponents o	f colum	ns, gross se	ction - flat		
Uniform compression in compo plates with both edges supported		umns, g	gross section	ı - flat		3.4.10.1
	$\mathbf{k}_1$	=	0.35			T3.3(D)
Max. distance between toes of fillets of supporting elements for plate	b'	=	17			
i.e. plate	t	=	1.5	mm		
Slenderness	b/t	=	11.33333			
			3			
Limit 1	S <sub>1</sub>	=	12.06			
Limit 2	$S_2$	=	49.94			
Factored limit state stress	φF <sub>L</sub>	=	104.50	MPa		
Most adverse compressive limit state stress	Fa	=	10.02	MPa		
Most adverse tensile limit state stress	Fa	=	104.50	MPa		
Most adverse compressive & Tensile capacity factor	f <sub>a</sub> /F <sub>a</sub>	=	0.01		PASS	
BENDING - IN-PLANE						
<b>3.4.15</b> Compression in beams, extubes, box sections	treme fibre,	gross s	section rectai	ngular		
Unbraced length for bending	$L_b$	=	1850	mm		

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							_
Second moment of area (weak axis)	ly	=	8945.75	mm <sup>4</sup>			ĺ
Torsion modulus	J	=	17744.20 6	mm³			
Elastic section modulus	Z	=	1141.05	${\sf mm}^3$			
Slenderness	S	=	335.10				
Limit 1	S <sub>1</sub>	=	21.80				
Limit 2	$S_2$	=	3854.05				
Factored limit state stress	φF∟	=	89.12	MPa		3.4.15(2)	
3.4.17 Compression in componen compression), gross section - flat							
	$\mathbf{k}_1$	=	0.5			T3.3(D)	
	$k_2$	=	2.04			T3.3(D)	
Max. distance between toes of fillets of supporting elements for plate	b'	=	17	mm			
·	t	=	1.5	mm			
Slenderness	b/t	=	11.33333				
Limit 1	S <sub>1</sub>	=	3 12.06				
Limit 2	S <sub>2</sub>	=	71.35				
	O <sub>2</sub>		7 1100				
Factored limit state stress	фГ∟	=	104.50	MPa			
Most adverse in-plane bending limit state stress	F <sub>bx</sub>	=	89.12	MPa			
Most adverse in-plane bending capacity factor	f <sub>bx</sub> /F <sub>bx</sub>	=	0.77		PASS		
BENDING - OUT-OF-PLANE							
NOTE: Limit state stresses, φF <sub>L</sub> and (doubly symmetric section)	re the same	for out	-of-plane bei	nding			
Factored limit state stress	φF <sub>L</sub>	=	89.12	MPa			
Most adverse out-of-plane bending limit state stress	F <sub>by</sub>	=	89.12	MPa			
Most adverse out-of-plane bending capacity factor	f <sub>by</sub> /F <sub>by</sub>	=	0.08		PASS		
COMBINED ACTIONS							
4.1.1 Combined compression and	bending					4.1.1(2)	1

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estored limit state stress From Shear for Stress From Shear for stress Adverseshear catter (Major Axis) at adverseshear catter (Minor Axis)  MBINED ACTIONS Combined Shear,	
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ess From Shear for st adverseshear ca for (Major Axis) st adverseshear ca	
ess From Shear for	
tored limit state str	
nderness	
ar web height	
s)	
.25 Shear in webs	
ess From Shear for	
ctored limit state str	
it 2	
it 1	
nderness	
ar web height	
<b>.24</b> Shear in webs s)	4.1.1(2
EAR	
	4.1.
	3.4.1
	3.4.1
	3.4.

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# 11.1.7 Centre Pole



**Job no.** 21-174-2 **Date**: 17/01/2022

NAME	SYMBOL		VALUE	UNIT	NOTES	REF
48x1.8	Centre Pole					
Alloy and temper	6063-T5					AS1664.1
	Ftu	=	152	MPa	Ultimate	T3.3(A)
Tension	F <sub>ty</sub>	=	110	MPa	Yield	
Compression	F <sub>cy</sub>	=	110	MPa		
01	F <sub>su</sub>	=	90	MPa	Ultimate	
Shear	$F_{sy}$	=	62	MPa	Yield	
Decrina	$F_bu$	=	317	MPa	Ultimate	
Bearing	$F_by$	=	179	MPa	Yield	
Modulus of elasticity	E	=	70000	MPa	Compressive	
	$k_{t}$	=	1.0			TO 4(D)
	kc	=	1.1			T3.4(B)
FEM ANALYSIS RESULTS						
Axial force	Р	=	0.516	kN	compression	
	Р	=	0	kN	Tension	
In plane moment	$M_{x}$	=	0	kNm		
Out of plane moment	$M_y$	=	0	kNm		
DESIGN STRESSES						
Gross cross section area	$A_g$	=	261.25485	$\text{mm}^2$		
In-plane elastic section modulus	$Z_{x}$	=	2908.7461	mm³		
Out-of-plane elastic section mod.	$Z_{y}$	=	2908.7461	mm³		
Stress from axial force	fa	=	P/A <sub>g</sub>			
		=	1.98	MPa	compression	
Others for a land		=	0.00	MPa	Tension	
Stress from in-plane bending	f <sub>bx</sub>	=	M <sub>x</sub> /Z <sub>x</sub> <b>0.00</b>	MDa	compression	
		=	0.00	MPa	compression	<u> </u>

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Stress from out-of-plane bending	$\mathbf{f}_{by}$	=	$M_y/Z_y$ <b>0.00</b>	MPa	compression	
Tension					,	
3.4.3 Tension in rectangular tube						3.4.3
	фГ∟	= OR	122.27	MPa		
	φFL	=	160.21	MPa		
COMPRESSION						
<b>3.4.8</b> Compression in columns, a 1. General	xial, gross	sectior	1			3.4.8.1
Unsupported length of member	L	=	400	mm		
Effective length factor	k	=	1.00			
Radius of gyration about buckling axis (Y)	r <sub>y</sub>	=	16.35	mm		
Radius of gyration about buckling axis (X)	$r_{x}$	=	16.35	mm		
Slenderness ratio	kLb/ry	=	24.47			
Slenderness ratio	kL/rx	=	24.47			
Slenderness parameter	λ	=	0.309			
	$D_c^*$	=	39.0			
	S <sub>1</sub> *	=	0.54			
	$S_2^*$	=	1.25			
	фсс	=	0.935			
Factored limit state stress	фГ∟	=	91.85	MPa		
2. Sections not subject to torsion	al or torsior	nal-flex	rural buckling			3.4.8.2
Largest slenderness ratio for flexural buckling	kL/r	=	24.47			
3.4.11 Uniform compression in c	omponents	of colu	ımns, gross s	section -		
flat plates Uniform compression in compon- plates with both edges, walls of r				- curved		3.4.11
plated mar boar dagod, mand or r	k <sub>1</sub>	=	0.35			T3.3(D)
mid-thickness radius of round tubular column or maximum	R <sub>m</sub>	=	23.1			, ,
mid-thickness radius	t	=	1.8	mm		
Slenderness	ι R <sub>m</sub> /t	=	12.833333	111111		
Limit 1	S <sub>1</sub>	_	1.69			
Limit 2	S <sub>1</sub>	=	672.46			
Limit 4	<b>U</b> 2	-	012.70			

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Factored limit state stress	φF <sub>L</sub>	=	103.88	MPa		
Most adverse compressive limit state stress	Fa	=	91.85	MPa		
Most adverse tensile limit state stress	Fa	=	122.27	MPa		
Most adverse compressive & Tensile capacity factor	f <sub>a</sub> /F <sub>a</sub>	=	0.02		PASS	
BENDING - IN-PLANE						
<b>3.4.13</b> Compression in beams, extubes	treme fibre	e, gros	s section rour	nd or oval		
Unbraced length for bending	L <sub>b</sub>	=	400	mm		
Second moment of area (weak axis)	ly	=	6.98E+04	mm <sup>4</sup>		
Torsion modulus	J	=	1.40E+05	$\rm mm^3$		
Elastic section modulus	Z	=	2908.7461	$\text{mm}^3$		
	R <sub>b</sub> /t	=	12.83			
Limit 1	S <sub>1</sub>	=	17.65			
Limit 2	S <sub>2</sub>	=	79.80			
Factored limit state stress	фГ∟	=	122.27	MPa		3.4.13
3.4.18 Compression in component edges supported	ts of beam	ıs - cı	ırverd plates ı	with both		
	<b>k</b> 1	=	0.5			T3.3(D)
	$k_2$	=	2.04			T3.3(D)
mid-thickness radius of round						
tubular column or maximum mid-thickness radius	$R_b$	=	23.1	mm		
	t	=	1.8	mm		
Slenderness	R <sub>b</sub> /t	=	12.833333			
Limit 1	S <sub>1</sub>	=	10.67			
Limit 2	$S_2$	=	79.80			
Factored limit state stress	фГ∟	=	101.17	MPa		
Most adverse in-plane bending limit state stress	F <sub>bx</sub>	=	101.17	MPa		
Most adverse in-plane bending capacity factor	f <sub>bx</sub> /F <sub>bx</sub>	=	0.00		PASS	
BENDING - OUT-OF-PLANE						

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NOTE: Limit state stresses, φF <sub>L</sub> (doubly symmetric section)	are the san	ne for o	ut-of-plane l	bending		
Factored limit state stress	φF∟	=	101.17	MPa		
Most adverse out-of-plane bending limit state stress	F <sub>by</sub>	=	101.17	MPa		
Most adverse out-of-plane bending capacity factor	f <sub>by</sub> /F <sub>by</sub>	=	0.00		PASS	
COMBINED ACTIONS						
4.1.1 Combined compression as	nd bending					4.1.1
	Fa	=	91.85	MPa		3.4.11
	Fao	=	103.88	MPa		3.4.11
	$F_bx$	=	101.17	MPa		3.4.18
	$F_by$	=	101.17	MPa		3.4.18
	f <sub>a</sub> /F <sub>a</sub>	=	0.022			
Check:	$f_a/F_a + f_{bx}/$	$F_{bx} + f_{by}$	$/F_{by} \leq 1.0$			4.1.1
i.e.	0.02	≤	1.0		PASS	
SHEAR						
<b>3.4.24</b> Shear in webs (Major Axis)						3.4.24
	R	=	24	mm		
	t	=	1.8	mm		
Equivalent h/t	h/t	=	29.58			
Limit 1	S <sub>1</sub>	=	33.38			
Limit 2	S <sub>2</sub>	=	59.31			
Factored limit state stress	φГ∟	=	58.90	MPa		
Stress From Shear force	$\mathbf{f}_{\mathbf{sx}}$	=	$V/A_w$			
3.4.25 Shear in webs (Minor			0.00	MPa		3.4.24
Axis)						3.4.24
Clear web height	R	=	24	mm		
	t	=	1.8	mm		
Equivalent h/t	h/t	=	29.58			
Factored limit state stress	φFL	=	58.90	MPa		
Stress From Shear force	$\mathbf{f}_{sy}$	=	$V/A_{w}$			
			0.00	MPa		

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Most adverseshear capacity factor (Major Axis)	f <sub>sx</sub> /F <sub>sx</sub>	=	0.00	MPa		
Most adverseshear capacity factor (Minor Axis)	$f_{sy}/F_{sy}$	=	0.00	Мра	PASS	
COMBINED ACTIONS						
<b>4.4</b> Combined Shear, Compresid		4.4				
Check:	$f_a/F_a + f_b/F$	$F_b + (f_s/F_s)$	$(s)^2 \le 1.0$			
i.e.	0.02	≤	1.0		PASS	

# 11.1.8 Summary Forces

MEMBER(S)	Section	b	d	t	Vx	Vy	Р	Mx	Му
		mm	mm	mm	kN	kN	kN	kN.m	kN.m
Post	120x85x3	85	120	3	-0	0.048	-0.524	0.8242	-0.2379
Cantilever Beam	60x35x3.5	35	60	3.5	-0.48	0.327	-0.057	-0.3859	0.1841
Brace 1	60x35x3.5	35	60	3.5	0.008	-0.375	-0.117	7.227E-19	0.2362
Brace 2	30x20x1.5	20	30	1.5	-0	0.026	-0.272	0	0.0154
Middle Beam	30x20x1.5	20	30	1.5	0.069	-0.012	0.162	-0.0477	0.0018
Corner Beam	30x20x1.5	20	30	1.5	-0.11	-0.026	0.201	-0.0788	-0.0062

MEMBER(S)	Section	d	t	Vx	Vy	P (Axial)	Mx	Му
		mm	mm	kN	kN	kN	kN.m	kN.m
Centre Pole	48x1.8	48	1.8	0	0	-0.516	0	0

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# 12 Appendix 'C' - Anchorage Design

ALL**FASTENERS** 



AFOS 2.0.3 (12012022) - Extended report

Company: Prime Consulting Engineers Pty. Ltd. E-mail: info@primeengineers.com.au

Designer: KZ Phone: 02 8964 1818

Designer: KZ Address: 21/1-7 Jordan St, Gladesville

Fax:

 Project:
 3.5m SQ Cantilever Umbrella
 Date:
 1/21/2022

 Comments:
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#### 1. Input Data

#### Selected anchors:

HLA-Z1 M10
 Sleeve anchor
 Zinc plated

Design based on AS 5216

- Assessment ETA-02/0030 (SZ) Issued by DIBt, on 9/13/2019
- · Effective anchorage depth het = 80 mm
- Drilled hole Φ x h<sub>0</sub> = 15.0 x 104 mm

#### Base material:

- Cracked concrete, Thickness of base material h=180mm Strength class 32MPa, f'c=32.0N/mm²
- Wide concrete reinforcement Rebar spacing a≥150mm for all Ø or a≥100mm for Ø≤10mm
- · No edge and stirrup reinforcement
- · Hammer drilled hole

#### Action loads:

· Predominantly static and quasi-static design loads

#### Installation:

- · Base plate lies on the concrete surface directly
- Without gap filling

#### Base plate:

- G250, E=200000N/mm<sup>2</sup>
   f<sub>y</sub>=250N/mm<sup>2</sup>, φ<sub>s</sub>=0.741, f<sub>yd</sub>= φ<sub>s</sub> · f<sub>y</sub>
- · Assumed: elastic plate
- Current thickness: 12.0mm σ/f<sub>yd</sub> =50.6/185.2=27.3%
- Circle

Diameter: 180 mm

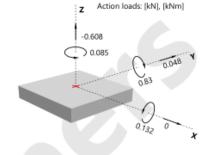
#### Profile:

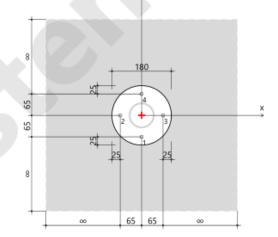
- Circular Hollow Section: 76.1x3.2 CHS H x W x T x FT [mm]: 76 x 76 x 3.2 x 0.0 Action point [mm]: [0, 0] Rotation counterclockwise: 0°
- · No profile stiffness

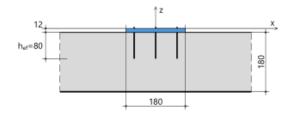
# Coordinates of anchors [mm]:

			Slotte	d hole
No.	X	y	L-x	L-y
1	0.0	-65.0		
2	-65.0	0.0		
3	65.0	0.0		
4	0.0	65.0		









Allfasteners Pty Ltd, 78-84 Logistics Street, Keilor Park, VIC 3042, Australia, Phone 1800 255349, www.allfasteners.com.au

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**Email:** <u>info@primeengineers.com.au</u> **Web:** <u>www.primeengineers.com.au</u>





## AFOS 2.0.3 (12012022) - Extended report

Company: Prime Consulting Engineers Pty. Ltd. E-mail: info@primeengineers.com.au

Designer: KZ Address: 21/1-7 Jordan St, Gladesville

 Address:
 21/1-7 Jordan St, Gladesville
 Fax:

 Project:
 3.5m SQ Cantilever Umbrella
 Date:
 1/21/2022

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#### Load cases, design load: [kN], [kNm]

Active	No.	Nz	V,	V <sub>y</sub>	Mz	Mx	My	Utilization	Decisive
•	1	-0.608	0.0	0.048	0.085	0.132	0.83	39.6%	
- 3	2	0.251	0.0	0.012	0.02	0.032	-0.62	ŝ i	

Phone:

02 8964 1818

# 2. Anchor internal forces [kN]

Tension load of anchors is calculated with elastic base plate.

Assumed: Anchor stiffness factor 0.50 → Anchor spring constant C<sub>g</sub> = 70.8 kN/mm.

Assumed: coefficient for concrete bedding factor b = 15.0 → concrete bedding factor Cc = b · fc = 480.0 N/mm³

Tension N <sub>i</sub>	Shear Vi	Shear x	Shear y
1.026	0.327	0.327	0.012
5.328	0.315	0.000	-0.315
0.000	0.339	0.000	0.339
2.116	0.327	-0.327	0.012
	1.026 5.328 0.000	1.026 0.327 5.328 0.315 0.000 0.339	1.026 0.327 0.327 5.328 0.315 0.000 0.000 0.339 0.000

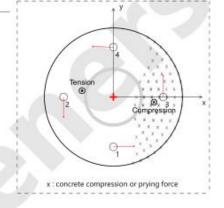
Maximum plate displacement into concrete (x/y=48.9/-10.4): 0.007 [mm]

Maximum concrete compressive stress: 3.31 [N/mm²] Mean concrete compressive stress: 1.23 [N/mm²] Resultant tension force in (x/y=-40.9/8.4): 8.470 [kN]

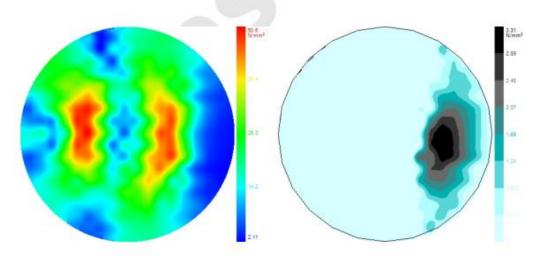
Resultant compression force in (x/y=53.1/-6.7): 9.078 [kN] Remark: The edge distance is not to scale.

Displacement and rotation of profile on base plate "
Displacement δ<sub>ε</sub> (+ve out of concrete): 0.031782 [mm]

Rotation  $\theta_{s}$ : 0.000190 [rad] Rotation  $\theta_{s}$ : 0.001089 [rad]



# Bending stresses in the base plate Concrete compression stresses under the base plate



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<sup>&</sup>quot; Calculated using the best fit plane





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Designer: KZ Address:

Project:

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## 3. Verification at ultimate limit state based on AS 5216

#### 3.1 Tension load

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure	2	5.328	30.667	17.4	√
Pull-out	2	5.328	13.440	39.6	√
Concrete cone failure	1,2,4	8.470	36.625	23.1	√
Concrete cone failure e *)		-			not applicable
Splitting failure	-	-	-	-	not applicable

<sup>1)</sup> additional proof for the fastening with elastic base plate

#### Steel failure

 $\beta_{N,s} = \, N^{\star} \, / \, N_{Rd,s}$  $N_{Rd,s} = N_{Rk,s} \cdot \varphi_{s,N}$ 

$N_{Rk,s}$	$\phi_{s,N}$	$N_{Rd,s}$	N*	$\beta_{N,s}$
[kN]		[kN]	[kN]	
46.0	0.667	30.667	5.328	0.174

#### Pull-out

N <sup>0</sup> Rkp	ψς	$\phi_{p,N}$	$N_{Rd,p}$	N*	$\beta_{N,p}$
[kN]			[kN]	[kN]	
16.0	1.26	0.667	13 440	5.328	0.396

## Concrete cone failure

N <sub>Rk,c</sub> =N <sup>0</sup> <sub>Rk,c</sub>	· ψ <sub>A,N</sub> · ψ <sub>s,1</sub>	ı · ψ <sub>re,N</sub> · ψ <sub>e</sub>	<sub>к,N</sub> - Фм,N	N <sup>0</sup> <sub>Rk,c</sub> =	k <sub>1</sub> · (f' <sub>c</sub> ) <sup>0.5</sup> ·	hef [N]	$\psi_{A,N} = A_c$	N/A° CN	$N_{Rd,c} = N_{Rk,c}$
N <sup>0</sup> Rk,c	$A_{c,N}$	$A^0_{cN}$	$\psi_{AN}$	$k_1$	$\varphi_{\varsigma,N}$	her	S <sub>cr,N</sub>	C <sub>cr,N</sub>	
FLA.ET	r 21	r 21				f	F	f 3	

N <sub>Rk.c</sub> [kN]	A <sub>cN</sub> [mm²]		ΨΑΝ	K <sub>1</sub>	ΦςΝ	n <sub>ef</sub> [mm]	s <sub>cr,N</sub> [mm]	C <sub>cr,N</sub> [mm]	
31.167	104400	57600	1.813	7.7	0.667	80.0	240.0	120.0	-

$\psi_{s,N}$	$\psi_{\text{re},N}$	e <sub>N.x</sub> [mm]	e <sub>N,y</sub> [mm]	ψес,N,x	<b>Ф</b> ес.N.у		$\psi_{MN}$	N <sub>Rk.c</sub> [kN]	N <sub>Rd,c</sub> [kN]	N* [kN]	$\beta_{N,c}$
1.0	1.0	19.2	8.4	0.862	0.935	0.806	1.207	54.937	36.625	8.470	0.231

Concrete cone failure for single anchor (additional proof for the fastening with elastic base plate) Verification is not required.

# Splitting

Verification of splitting failure is not necessary, because:

# 3.2 Shear

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure (without I. arm)	3	0.339	38.400	0.9	√
Pry-out	3	0.339	21.509	1.6	√
Concrete edge failure					not applicable

## Steel failure without lever arm

$V_{Rd,s} = V_i $	$Rk_s \cdot k_7 \cdot \varphi_{s,V}$	$\beta_{V,s} = V$	/* / V <sub>Rd,s</sub>		
$V_{Rk,s}$	k <sub>7</sub>	$\varphi_{s,V}$	$V_{\text{Rd,s}}$	V*	$\beta_{V,s}$
[kN]			[kN]	[kN]	
48.0	1.0	0.8	38.400	0.339	0.009

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<sup>•</sup> The smallest edge distance of anchor is  $c \ge 1.2 c_{cr,sp}$  .



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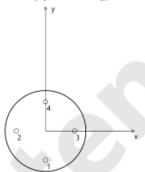
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#### Pry-out failure

N <sub>Rk,c</sub> =N <sup>0</sup> <sub>Rk,c</sub>	· ψ <sub>A,N</sub> · ψ <sub>s,1</sub>	N·Ψ <sub>re,N</sub> ·Ψ <sub>e</sub>	c,V,cp N <sup>0</sup>	$k_{k,c} = k_1 \cdot (f_c)$	0.5 · h <sub>ef</sub> 1.5 [N	V) Ψ <sub>A,N</sub>	$=A_{c,N}/A_{c,N}^0$	$V_{Rk,cp} = I$	t <sub>8</sub> · N <sub>Rk,c</sub>	$V_{Rd,cp} = V_{Ri}$	<sub>k,cp</sub> · φ <sub>cp,V</sub>
N <sup>0</sup> <sub>Rk,c</sub> [kN]	A <sub>c,N</sub> [mm <sup>2</sup> ]	A <sup>0</sup> c,N [mm²]	ΨΑΝ	$\psi_{s,N}$	$\psi_{\text{re},N}$	h <sub>ef</sub> [mm]	s <sub>cr,N</sub> [mm]	c <sub>cr,N</sub> [mm]	k <sub>1</sub>	k <sub>8</sub>	$\varphi_{cp,V}$
31.167	29813	57600	0.518	1.0	1.0	80.0	240.0	120.0	7.7	2.0	0.667
e <sub>V,cp,x</sub> [mm]	e <sub>V,cp,y</sub> [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	N <sub>Rk,c</sub> [kN]	V <sub>Rk,cp</sub> [kN]	V <sub>Rd,cp</sub> [kN]	V* [kN]	$\beta_{V,cp}$		
0.0	0.0	1.0	1.0	1.0	16.132	32.264	21.509	0.339	0.016		

# Related area for calculation of pry-out failure $A_{c,N}$ :



## Concrete edge failure

Verification for concrete edge failure is not necessary, because there is no concrete edge.

#### 3.3 Combined tension and shear

	Anchor		r	Tension( β <sub>N</sub> )	Shear( $\beta_V$ )	Condition	Utilization [%]	Status	
Steel		2		0.174	0.008	$\beta^{2}_{N} + \beta^{2}_{V} \le 1.0$	3.0	√	
Concrete		2		0.396	0.015	$\beta^{1.5}_{N} + \beta^{1.5}_{V} \le 1.0$	25.1	√	

# Anchor-related utilization

A-No.	β <sub>N,s</sub>	β <sub>N,o</sub>	$\beta_{N,c}$	$\beta_{N,ec}$	$\beta_{N,s_D}$	β <sub>v,s</sub>	$\beta_{\text{V,cp}}$	$\beta_{V,c}$	β <sub>N,c,max,E</sub>	$\beta_{\text{V,c,max,E}}$	$\beta_{combi,c,E}$	$\beta_{\text{combi,s,E}}$
1	0.033	0.076	0.231	0.000	0.000	0.009	0.015	0.000	0.231	0.015	0.113	0.001
2	0.174	0.396	0.231	0.000	0.000	0.008	0.015	0.000	0.396	0.015	0.251	0.030
3	0.000	0.000	0.000	0.000	0.000	0.009	0.016	0.000	0.000	0.016	0.002	0.000
4	0.069	0.157	0.231	0.000	0.000	0.009	0.015	0.000	0.231	0.015	0.113	0.005

BNC/NEXE: Highest utilization of individual anchors under tension loading except steel failure

 $\beta_{V,c,ma,E} \colon Highest \ utilization \ of \ individual \ anchors \ under \ shear \ loading \ except \ steel \ failure$ 

 $\beta_{combined} : Utilization of individual anchors under combined tension and shear loading except steel failure$ 

Beambis, E: Utilization of individual anchors under combined tension and shear loading at steel failure

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4. Displacement

**Tension loading:**  $N_k^h = N^{*h} / 1.4$ Short-term displacement:  $\delta_N^0 = (N_k^h / N_0) \cdot \delta_{N0}$ Long-term displacement:  $\delta_N^{\infty} = (N_k^h / N_0) \cdot \delta_{N\infty}$ 

Shear loading: Short-term displacement: Long-term displacement:  $V_k^h = V_k^h / 1.4$   $\delta_V^0 = (V_k^h / V_0) \cdot \delta_{V0}$  $\delta_V^\infty = (V_k^h / V_0) \cdot \delta_{V\infty}$ 

 $N^{*h}$   $N_0$   $\delta_{N0}$   $\delta_{N\infty}$  [kN] [kN] [mm] [mm] 5.328 7.6 0.5 1.3

δ<sub>N</sub> δ<sub>N</sub> [mm] [mm] **0.250 0.651** 

V\*<sup>h</sup> V<sub>0</sub> δ<sub>v0</sub> [kN] [kN] [mm] 0.339 27.5 3.6 δ<sub>√∞</sub> δ<sub>√</sub><sup>0</sup>
[mm] [mm]

δ<sub>v</sub><sup>∞</sup> [mm]

#### 5. Remarks

 Capacity verifications of Section 3 are in accordance with AS 5216. For more complex cases which are outside of AS 5216, the same principles of AS 5216 are still used.

- For connections with a flexurally rigid base plate, it is assumed that the base plate is sufficiently rigid. However, the current anchor
  design methods (ETAG, Eurocode, AS 5216, ACI 318, CSA A23.3) do not provide any usable guidance to check for rigidity. In the
  realistically elastic (flexible) base plate, the tension load distribution between anchors may be different to that in the assumed rigid
  base plate. The plate prying effects could further increase anchor tension loading. To verify the sufficient base plate bending
  rigidity, the stiffness condition according to the publication "Required Thickness of Flexurally Rigid Base plate for Anchor
  Fastenings" (fib Symposium 2017 Maastricht) is used in this software.
- For connections with an elastic base plate, the anchor tension forces are calculated with the finite element method with
  consideration of deformations of base plate, anchors and concrete. Background for design with elastic base plates is described in
  the paper "Design of Anchor Fastenings with Elastic Base Plates Subjected to Tension and Bending". This paper was published in
  "Stahlbau 88 (2019), Heft 8" and "5. Jahrestagung des Deutschen Ausschusses für Stahlbeton DAfStb 2017".
   Anchor shear forces are calculated with the assumption of a rigid base plate. Attention should be paid to a narrow base plate with
  a width to length ratio of less than 1/3.
- Verification for the ultimate limit state and the calculated displacement under service working load are valid only if the anchors are installed properly according to ETA.
- For design in cracked concrete, anchor design standards/codes assume that the crack width is limited to ≤ 0.3mm by
  reinforcement. Splitting failure in cracked concrete is prevented by this reinforcing. The user needs to verify that this reinforcing is
  present in cracked concrete. Generally, concrete structures design standards/codes (e.g. AS 3600) meet this crack width
  requirement for most structures. Particular caution must be taken at close edge distances where the location of reinforcing is not
  clearly known.
- Verification of strength of concrete elements to loads applied by fasteners is to be done in accordance with AS 5216.
- All information in this report is for use of Allfasteners products only. It is the responsibility of the user to ensure that the latest
  version of the software is used, and in accordance with AFOS licensing agreement. This software serves only as an aid to interpret
  the standards and approvals without any guarantee to the absence of errors. The results of the software should be checked by a
  suitably qualified person for correctness and relevance of the results for the application.

The load-bearing capacity of the anchorage is: verified!

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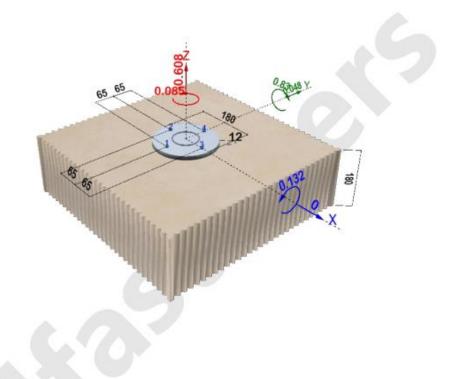
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# Anchorage figure in 3D:



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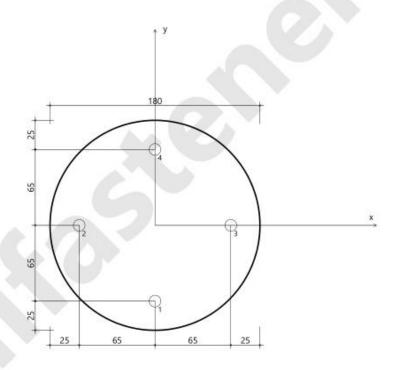
Anchor: HLA-Z1 M10

Drilled hole:  $d_0 \times h_0 = 15 \times 104 \text{ mm}$ 



Base plate: G250

Thickness: t = 12 mmClearance hole:  $d_f = 17 \text{ mm}$ 



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