APOLLO CRADLES LTD
X-BEAM 15° RIDGE FRAME
DESIGN CHECK CALCULATIONS

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R0151-001

Brief

The brief is to check the capacity of the Apollo ridge frame unit when utilised with xbeams in a temporary roof

The beams are manufactured from tube extrusions in aluminium alloy 6082 T6.

Alloy

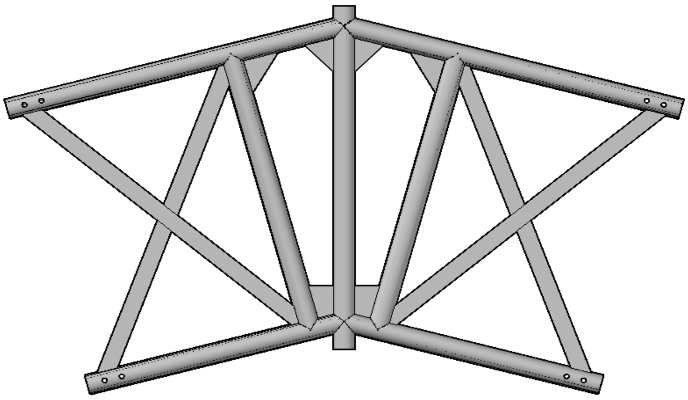
The alloy used is 6082 T6:

$f_o =$ 250 N/mm^2

$f_u =$ 290 N/mm^2

Layout

The geometry of the beam is shown in the drawing below:



REF: Drawing No R0151/002

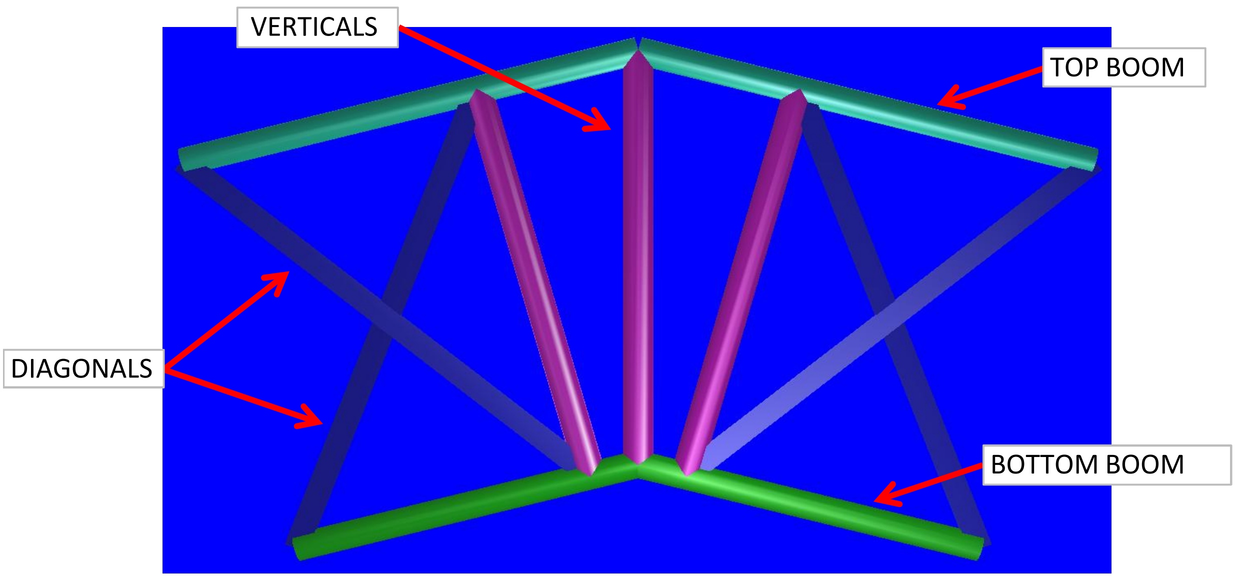
Design

Eurocode 9: Design of Aluminium structures — EN 1999-1-1

Alloy used is 6082 T6 throughout

STRAP Model

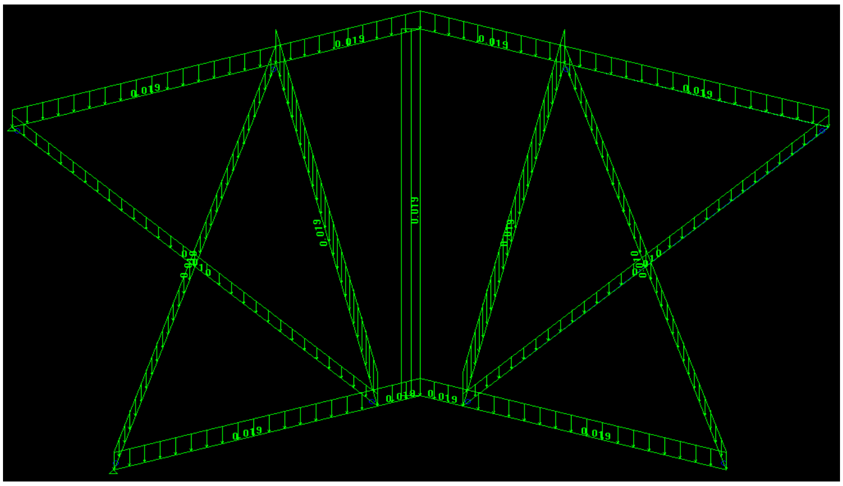
The structure was analysed in STRAP structural analysis program.



Load Cases

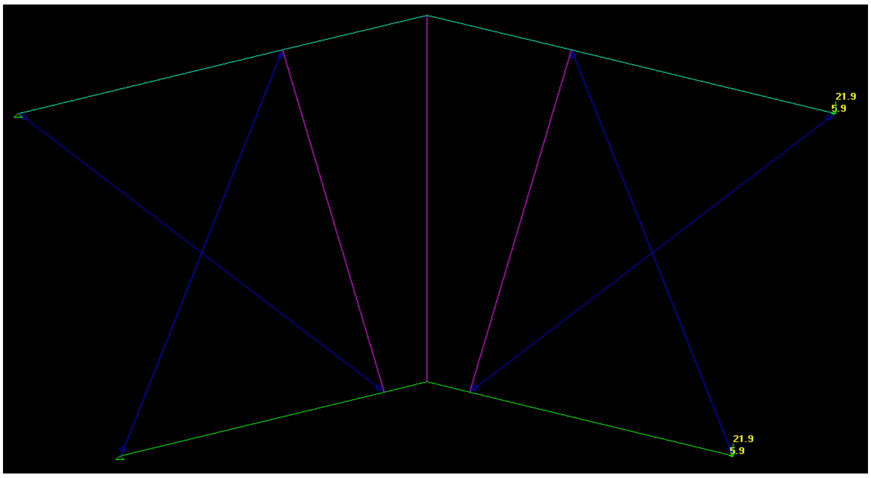
Load Case 1 — Self Weight

Self weight of all members is factored by 1.15 to account for connections



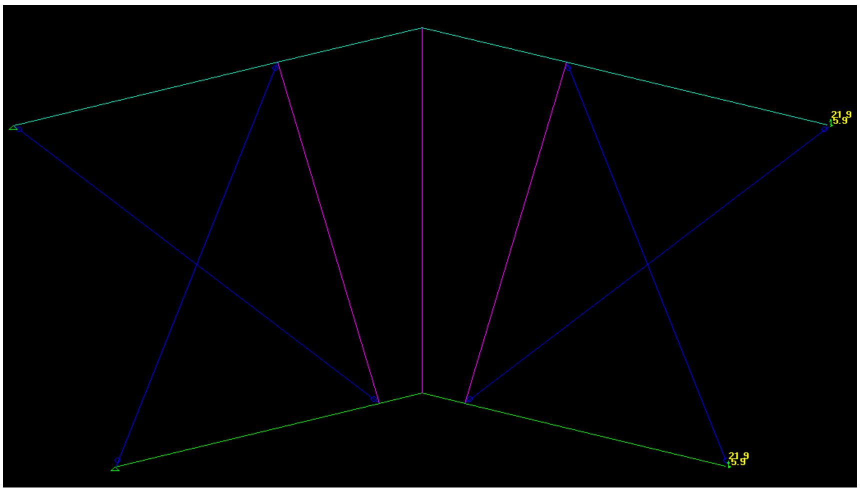
Load Case 2 — Beam shear downforce

The max end shear from an Xbeam is applied locally to the ridge frame



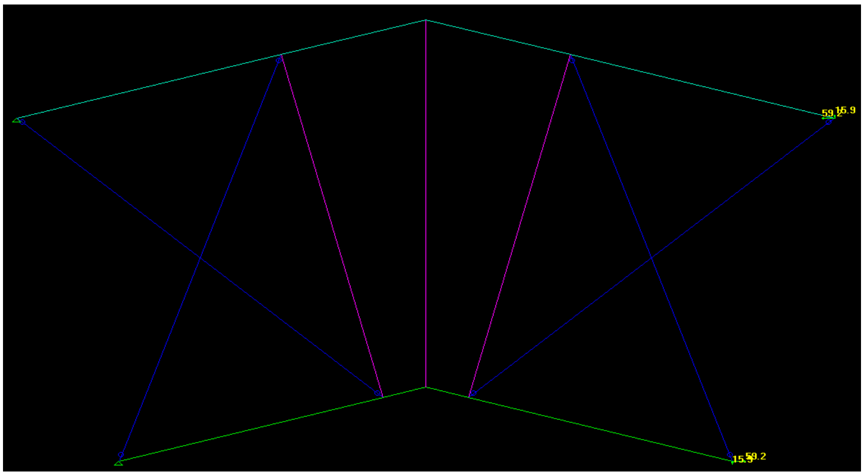
Load Case 3 — Beam shear uplift

The max end shear from an Xbeam is applied locally to the ridge frame



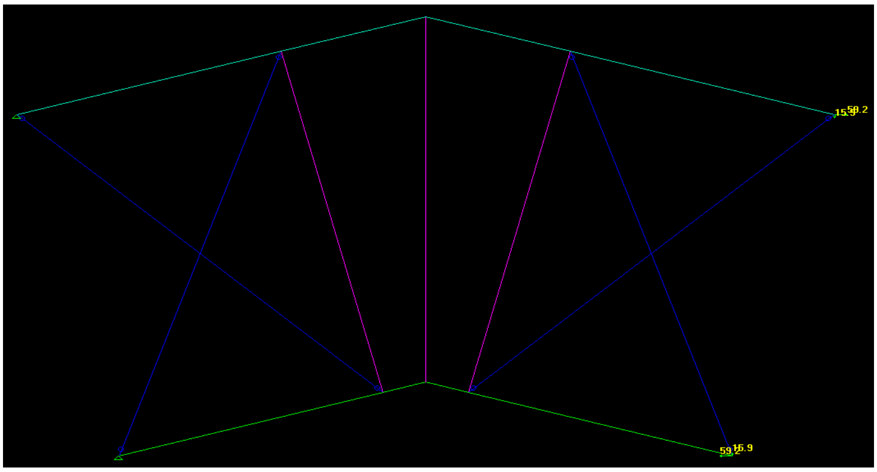
Load Case 4 Beam moment downforce

The push/pull loads from Xbeam bending moment is applied to the ridge frame



Load Case 5 Beam moment uplift

The push/pull loads from Xbeam bending moment is applied to the ridge frame



Load Combinations

Combination Number	Combination Description	Load Cases
1	SLS Downforce	1+2+4
2	ULS Downforce	1+2+4
3	SLS Uplift	1+3+5
4	ULS Uplift	1+3+5

Above Combinations were checked for the following design factors:

γ_D= 1.25

γ_{Shear}= 0.5

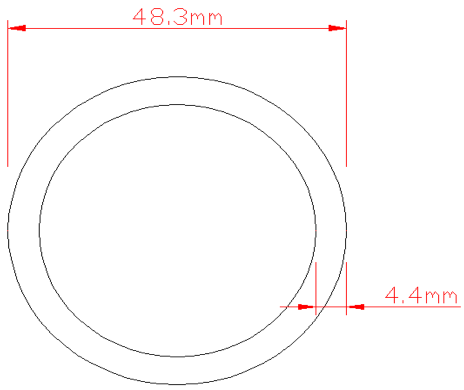
$\gamma_{Bending}$= 1.50

Due to the nature of the loading on a roof, the maximum shear will be found at the support with a very low shear force found in the ridge itself.

CALCULATION SHEET	Project :	Apollo Ridge Frame to Eurocode			ALAN WHITE DESIGN
	Element :	Main Boom Capacity			
	Job Number :	R0151	By : eas	Date:Sep-12	
	Document No :	001	Checked :anw	Date:Sep-12	

CHS Boom Layout

Boom 48.3 x 4.4 CHS



Section Properties

A=	607	mm^2
I=	147654	mm^4
W_{el}=	6114	mm^3
W_{pl}=	8254	mm^3
r_y=	15.6	mm

for slenderness

β=	b/t	b=	48.3
=	10.98	t =	4.4
ε=	sqrt(250/f_o)	f_o=	250N/mm^2
=	1.00		

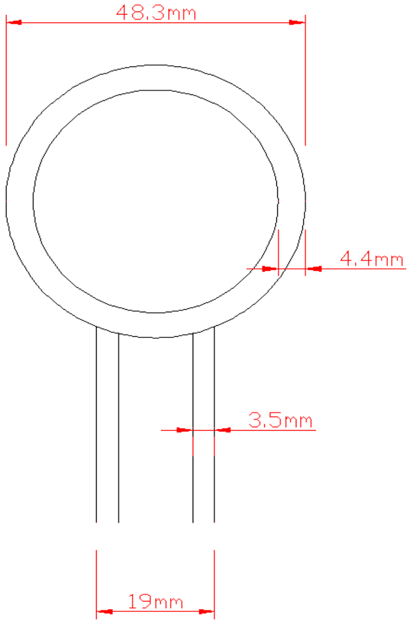
Class A, without welds, Internal parts

β_1=	11ε
=	11*1.0
=	11.00
>	10.98

Section is class 1

Boom 48.3 x 4.4 CHS HAZ

There is a HAZ at welded joint to the diagonal brace



t_{boom}= 4.40mm

$t_{diagonal}$= 3.5mm

$t_{average}$= 3.95mm

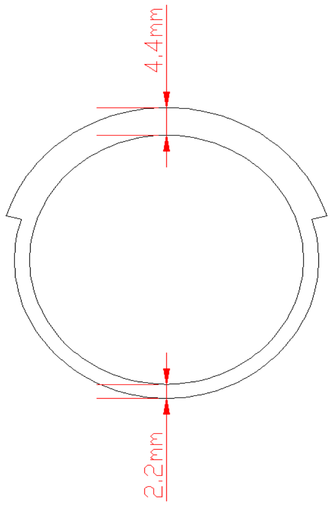
All welds are TIG.
As per EN 1999-1-1 6.1.6.3

b_{haz} = 30mm

Therefore HAZ extends 30mm from intersection of welded materials

HAZ Section Layout

Take section shown as non-HAZ.



Revised Section Properties

$A=$ 418 mm^2
$I=$ 92785 mm^4
$W_{el}=$ 3398 mm^3
$W_{pl}=$ 4587 mm^3
$r_y=$ 14.7 mm

Truss Boom Moment Capacity (6.2.5.1)

$M_{c,Rd} = \alpha W_{el} f_o / \gamma_{M1}$

$\alpha = W_{pl}/W_{el}$ (Table 6.4)
$= 1.35$
$W_{el} =$ 3.40 cm^3
$f_o =$ 250 N/mm^2
$\gamma_{M1} =$ 1.1 (6.1.3)

= 1.35*3.40*250/1100
$M_{c,Rd} =$ 1.04 kNm

Truss Boom Shear Capacity (6.2.6)

$V_{Rd} = A_v f_o / \sqrt{3}\gamma_{M1}$

$A_v=$ 0.6A
$A_v=$ 0.6*607
$A_v=$ 364.20 mm^2
$\gamma_{M1}=$ 1.1
$f_o =$ 250 N/mm^2

= 364.20*250/(SQRT(3)*1100)
$V_{Rd} =$ 47.79 kN

Truss Top Boom Axial Comp Capacity @ 560mm (effective length of beam between restraints)

$N_{b,Rd} = k \times A_{eff} f_o / \gamma_{M1}$ (6.3.1.1)

$N_{cr} = \pi^2 EI / k^2 L^2$ (Appendix I.3)

$E = 70{,}000 \ N/mm^2$

$I = 147{,}654 \ mm^4$

$k = 0.50$ (Table I.2)

$L = 560 \ mm$

$N_{cr} =$ ((PI()^2)*70000*147654)/((0.5^2)*(560^2))

$= 1{,}301{,}148.72 \ N$

$\lambda = \sqrt{A_{eff} f_o / N_{cr}}$ (6.3.1.2)

$= 0.34$ $A_{eff} = 607 \ mm^2$

$X = 1/ \Phi+\sqrt{\Phi^2-\lambda^2}$

$\Phi = 0.5(1+\acute{\alpha}(\lambda-\lambda_o)+\lambda^2)$

$\acute{\alpha} = 0.20$ Table 6.6

$\lambda_o = 0.10$ Table 6.6

$\Phi = 0.58$

$X = 0.948$

$k = 1-(1-(A_1/A)10^{-\lambda}-(0.005+0.1(A_1/A))\lambda^{1.3(1-\lambda)}$

$A_1 = A-A_{HAZ}(1-p_{o,HAZ})$

= 607-189*(1-0.5)

$= 512.50 \ mm^2$

$k = 0.875$

$N_{b,Rd} = k \times A_{eff} f_o / \gamma_{M1}$

$N_{b,Rd} =$ 0.875*0.948*607*250/1100

= 114.43 kN

Truss Bottom Boom Axial Comp Capacity @ 1100mm (effective length of beam between restraints)

$N_{b,Rd} = k \times A_{eff} f_o / \gamma_{M1}$ (6.3.1.1)

$N_{cr} = \pi^2 EI / k^2 L^2$ (Appendix I.3)

$E = 70{,}000 \ N/mm^2$

$I = 147{,}654 \ mm^4$

$k = 0.50$ (Table I.2)

$L = 1{,}100 \ mm$

$N_{cr} =$ ((PI()^2)*70000*147654)/((0.5^2)*(1100^2))

$= 337{,}223.34 \ N$

$\lambda = \sqrt{A_{eff} f_o / N_{cr}}$ (6.3.1.2)

$= 0.67$ $A_{eff} = 607 \ mm^2$

$X = 1/ \Phi+\sqrt{\Phi^2-\lambda^2}$

$\Phi = 0.5(1+\acute{\alpha}(\lambda-\lambda_o)+\lambda^2)$

$\acute{\alpha} =$ 0.20 Table 6.6

$\lambda_o =$ 0.10 Table 6.6

$\Phi =$ 0.78

$X =$ 0.844

$k = 1-(1-(A_1/A)10^{-\lambda}-(0.005+0.1(A_1/A))\lambda^{1.3(1-\lambda)}$

$A_1 = A-A_{HAZ}(1-\rho_{o,HAZ})$

= 607-189*(1-0.5)

= 512.50 mm^2

k= 0.853

$N_{b,Rd} = k \times A_{eff} f_o / \gamma_{M1}$

$N_{b,Rd} =$ 0.853*0.844*607*250/1100

= 99.32 kN

Truss Boom Axial Tension Capacity (6.2.3)

1. General yielding

$N_{o,Rd} = A_g f_o / \gamma_{M1}$

$f_o =$ 250N/mm^2

$A_g =$ 607mm^2

$\gamma_{M1} =$ 1.1

= 607*250/1100

= 137.95 kN

2. Local failure

$N_{u,Rd} = A_{net} f_u / \gamma_{M2}$

$f_u =$ 290N/mm^2

$A_{net} = \rho_{u,haz} * A$

= 0.64*607mm^2

= 388.5mm^2

$\gamma_{M1} =$ 1.25

= 388.5*290/1250

= 90.13 kN

Gusset plate

A 80mm x 12.7mm thick gusset plate is added to strengthen the top and bottom boom
Assume section fully heat affected

A = 80*12.7
= 1,016.00 mm^2

W_{el} = 12.7*80^2/6000
= 13.55 cm^3

$M_{u,Rd}$ = $W_{net}\, f_u / \gamma_{M2}$

W_{net} = $W_{el} * \rho_{u,\, haz}$
= 13.55*0.64
= 8.67 cm^3
f_u = 290 N/mm^2
γ_{M2} = 1.25 (6.1.3)

= 8.67*290/1250
$M_{u,Rd}$ = 2.01 kNm

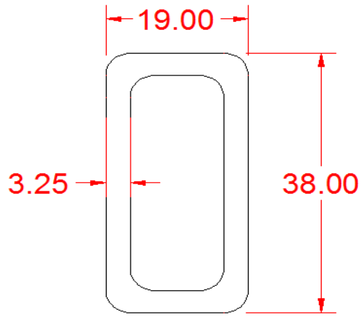
Gusset Plate Shear Capacity (6.2.6)

V_{Rd} = $A_v\, f_o / \sqrt{3}\gamma_{M1}$

A_v= A
A_v= 1,016 mm^2
γ_{M1}= 1.1
f_o = 250 N/mm^2

= 1016*250/(SQRT(3)*1100)
V_{Rd} = 133.32 kN

RHS-Oval Diagonal Member
Diagonal 38 x 19 x 3.25mm Oval



Section Properties

A=	328	mm^2
I=	53341	mm^4
W_{el}=	2807	mm^3
W_{pl}=	3729	mm^3
r_y=	7.0	mm

for slenderness β= b/t b= 38-2*3.25

= 31.50

= 9.69 t = 3.25

ε= sqrt(250/f_o) f_o= 250N/mm^2

= 1.00

Class A, without welds, Internal parts β_1= 11ε

= 11*1.0

= 11.00

> 9.69

Section is class 1

Diagonal Axial Comp Capacity @ 400mm (effective length of beam)

$N_{b,Rd}$ = k x A_{eff} f_o /γ_{M1} (6.3.1.1)

N_{cr} = π^2 EI / k^2 L^2 (Appendix I.3)

E = 70,000 N/mm^2

I = 53,341 mm^4

k = 0.50

L = 400 mm

N_{cr} = ((PI()^2)*70000*53341)/((0.5^2)*(400^2))

= 921,295.49 N

λ = $\sqrt{A_{eff}\, f_o / N_{cr}}$ (6.3.1.2)

= 0.30 A_{eff} = 328 mm^2

$X = 1/\ \Phi+\sqrt{\Phi^2-\lambda^2}$

$\Phi = 0.5(1+\acute{\alpha}(\lambda-\lambda_o)+\lambda^2)$

$\acute{\alpha} =$ 0.20 Table 6.6

$\lambda_o =$ 0.10 Table 6.6

$\Phi =$ 0.56

$X =$ 0.89

$A_1 = A - A_{HAZ}(1-p_{o,HAZ})$

= 328-164*(1-0.5)

= 246.00 mm^2

k= 0.833

$N_{b,Rd}$ = 0.833*0.89*328*250/1100

= 55.27 kN

Diagonal Axial Tension Capacity (6.2.3)

1. General yielding

$N_{o,Rd} = A_g\, f_o / \gamma_{M1}$

$f_o =$ 250N/mm^2

$A_g =$ 328mm^2

$\gamma_{M1} =$ 1.1

= 328*250/1100

= 79.55 kN

2. Local failure

$N_{u,Rd} = A_{net}\, f_u / \gamma_{M2}$

$f_u =$ 290N/mm^2

$A_{net} = \rho_{u,haz} * A$

= 0.64*328mm^2

= 209.9mm^2

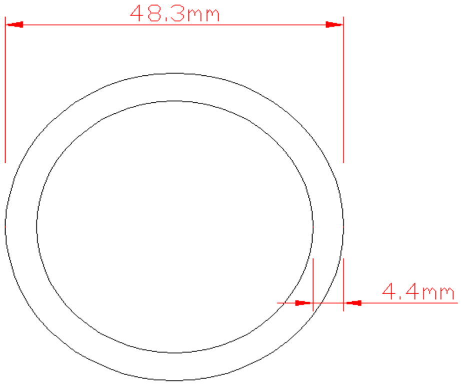
$\gamma_{M1} =$ 1.25

= 209.9*290/1250

= 48.70 kN

Vertical 48.3 x 4.4mm CHS

Vertical CHS Layout



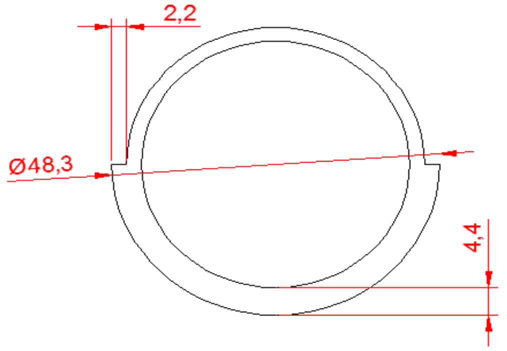
Section Properties

	$A=$	607	mm^2	
	$I=$	147654	mm^4	
	$W_{el}=$	6114	mm^3	
	$W_{pl}=$	8254	mm^3	
	$r_y=$	15.6	mm	
for slenderness	$\beta=$	b/t		b= 48.3
	=	10.98		t = 4.4
	$\varepsilon=$	$\text{sqrt}(250/f_o)$		$f_o=$ 250N/mm^2
	=	1.00		
Class A, without welds, Internal parts	$\beta_1=$	11ε		
	=	11*1.0		
	=	11.00		
	>	10.98		

Section is class 1

Vertical HAZ at Gusset Plate

Vertical CHS Layout



Section Properties

A=	447	mm^2
I=	105237	mm^4
W_{el}=	4358	mm^3
W_{pl}=	5883	mm^3
r_y=	15.3	mm

Vertical CHS Moment Capacity (6.2.5.1)

$M_{c,Rd}$ = $\alpha W_{el} f_o / \gamma_{M1}$ (Table 6.4)

α =	1.35	
W_{el} =	4.36	cm^3
f_o =	250	N/mm^2
γ_{M1} =	1.1	(6.1.3)

= 1.35*4.36*250/1100

$M_{c,Rd}$ = 1.34 kNm

Vertical CHS Shear Capacity (6.2.6)

V_{Rd} = $A_v f_o / \sqrt{3}\gamma_{M1}$

A_v=	0.6A	
A_v=	0.6*607	
A_v=	364.20	mm^2
γ_{M1}=	1.1	
f_o =	250	N/mm^2

= 364.20*250/(SQRT(3)*1100)

V_{Rd} = 47.79 kN

Vertical CHS Axial Comp Capacity @ 500mm (effective length of beam)

$N_{b,Rd}$ = $k \times A_{eff} f_o / \gamma_{M1}$ (6.3.1.1)

N_{cr} = $\pi^2 EI / k^2 L^2$ (Appendix I.3)

E = 70,000 N/mm^2

I = 105,237 mm^4

k = 0.50

L = 500 mm

N_{cr} = ((PI()^2)*70000*105237)/((0.5^2)*(500^2))

= 1,163,285.27 N

λ = $\sqrt{A_{eff} f_o / N_{cr}}$ (6.3.1.2)

= 0.36 A_{eff} = 607 mm^2

X = $1/ \Phi+\sqrt{\Phi^2-\lambda^2}$

$\Phi = 0.5(1+\acute{\alpha}(\lambda-\lambda_o)+\lambda^2)$

$\acute{\alpha}$ = 0.20 Table 6.6

λ_o= 0.10 Table 6.6

Φ = 0.59

X = 0.85

k= $1-(1-(A_1/A)10^{-\lambda}-(0.005+0.1(A_1/A))\lambda^{1.3(1-\lambda)}$

A_1= $A-A_{HAZ}(1-p_{o,HAZ})$

= 607-160*(1-0.5)

= 527.00 mm^2

k= 0.639

$N_{b,Rd}$ = 0.639*607*250/1100

= 88.15 kN

Vertical CHS Axial Tension Capacity (6.2.3)

1. General yielding $N_{o,Rd}$ = $A_g f_o / \gamma_{M1}$

f_o = 250N/mm^2

A_g = 607mm^2

γ_{M1} = 1.1

= 607*250/1100

= 137.95 kN

2. Local failure $N_{u,Rd}$ = $A_{net} f_u / \gamma_{M2}$

f_u = 290N/mm^2

A_{net} = A* $\rho_{u,haz}$

= 607*0.64

= 388.5mm^2

γ_{M1} = 1.25

= 388.5*290/1250

= 90.13 kN

Results	Action	Formula	Ultimate	Calculated	Factor

Arrangement: 15 Degree ridge
Load orientation: Downforce

Top Boom Capacity

	Action	Formula	Ultimate	Calculated	Factor
	Moment	$M_{Rd,x}$	1.04	0.58	0.56
	Shear	V_{Rd}	47.79	2.43	0.05
	Tension	$N_{u,Rd}$	90.13	0.00	0.00
	Compression	$N_{b,Rd}$	114.43	73.30	0.64
	Combined	< 1.0	1.00	0.93	0.93

Bottom Boom Capacity

	Action	Formula	Ultimate	Calculated	Factor
	Moment	$M_{Rd,x}$	3.05	1.37	0.45
	Shear	V_{Rd}	181.10	20.28	0.11
	Tension	$N_{u,Rd}$	90.13	83.11	0.92
	Compression	$N_{b,Rd}$	99.32	0.00	0.00
	Combined	< 1.0	1.00	0.25	0.25

Diagonal Capacity

	Action	Formula	Ultimate	Calculated	Factor
	Tension	$N_{u,Rd}$	48.70	15.49	0.32
	Compression	$N_{b,Rd}$	55.27	14.30	0.26

Vertical Capacity

	Action	Formula	Ultimate	Calculated	Factor
	Moment	$M_{Rd,x}$	1.34	0.71	0.53
	Shear	V_{Rd}	47.79	1.66	0.03
	Tension	$N_{u,Rd}$	90.13	36.24	0.40
	Compression	$N_{b,Rd}$	88.15	16.22	0.18
	Combined	< 1.0	1.00	0.45	0.45

Results	Action	Formula	Ultimate	Calculated	Factor

Arrangement: 15 Degree ridge
Load orientation: Uplift

Top Boom Capacity

Moment	$M_{Rd,x}$	1.04	0.65	0.62
Shear	V_{Rd}	47.79	2.42	0.05
Tension	$N_{u,Rd}$	90.13	83.80	0.93
Compression	$N_{b,Rd}$	114.43	0.00	0.00
Combined	< 1.0	1.00	0.44	0.44

Bottom Boom Capacity

Moment	$M_{Rd,x}$	3.05	0.66	0.22
Shear	V_{Rd}	181.10	20.17	0.11
Tension	$N_{u,Rd}$	90.13	0.00	0.00
Compression	$N_{b,Rd}$	99.32	83.12	0.84
Combined	< 1.0	1.00	0.87	0.87

Diagonal Capacity

Tension	$N_{u,Rd}$	48.70	14.28	0.29
Compression	$N_{b,Rd}$	55.27	15.44	0.28

Vertical Capacity

Moment	$M_{Rd,x}$	1.34	0.70	0.52
Shear	V_{Rd}	47.79	1.65	0.03
Tension	$N_{u,Rd}$	90.13	9.85	0.11
Compression	$N_{b,Rd}$	88.15	36.32	0.41
Combined	< 1.0	1.00	0.65	0.65

Summary

All structural members have passed design checks for specified loading

The ridge member has been checked and found to be stronger than the x-beams to which it will be connected.

The ridge can be used as part of an x-beam roof which is properly laced and braced without additional calculation.