X0069-001A
APOLLO CRADLES LTD
APOLLO X-BEAM 1.0m WALL ATTACHMENT
DESIGN CHECK CALCULATIONS

Alan N White B.Sc., M.Eng., C.Eng., M.I.C.E., M.I.H.T.

Malachy Ryan B.Eng, M.Sc., C.Eng., M.I.C.E.

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17-19 Hill Street
Kilmarnock
KA3 1HA
Tel:01563 594 621
Fax:01563 593 056
enquiry@alanwhitedesign.com

Document Revision History

Revision	Description	Author	Revision Date	Checked
A	Initial Issue	ANW	13/07/17	MMR

Brief

The brief is to check the capacity of the Apollo 1.0m X-beam Wall Attachment when utilised to support an outstand platform.

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

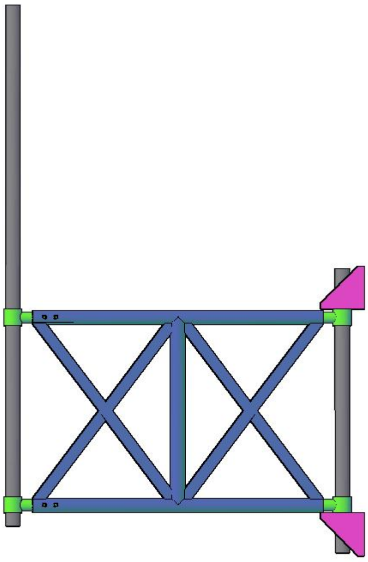
Layout

The layout of the X-beam attachment is shown below, configured for testing

These outstands are attached to a vertical surface at usually 1m intervals to support a platfom

The 1m long section of X-beam is connected by welded couplers to a support tube, which is held in place by brackets, bolted to the wall

The other end of the beam holds a vertical tube to support the guardrail.



Design

Eurocode 9: Design of Aluminium structures	EN 1999-1-1
Eurocode 3: Design of steel structures	EN 1993-1-1
NASC Technical guidance on the design of scaffolding to BS EN 12811-1	TG20

Assumptions

The frames are spaced at 1.00m centres.

The structure to which the frame is attached will be capable of carrying the applied support loads.

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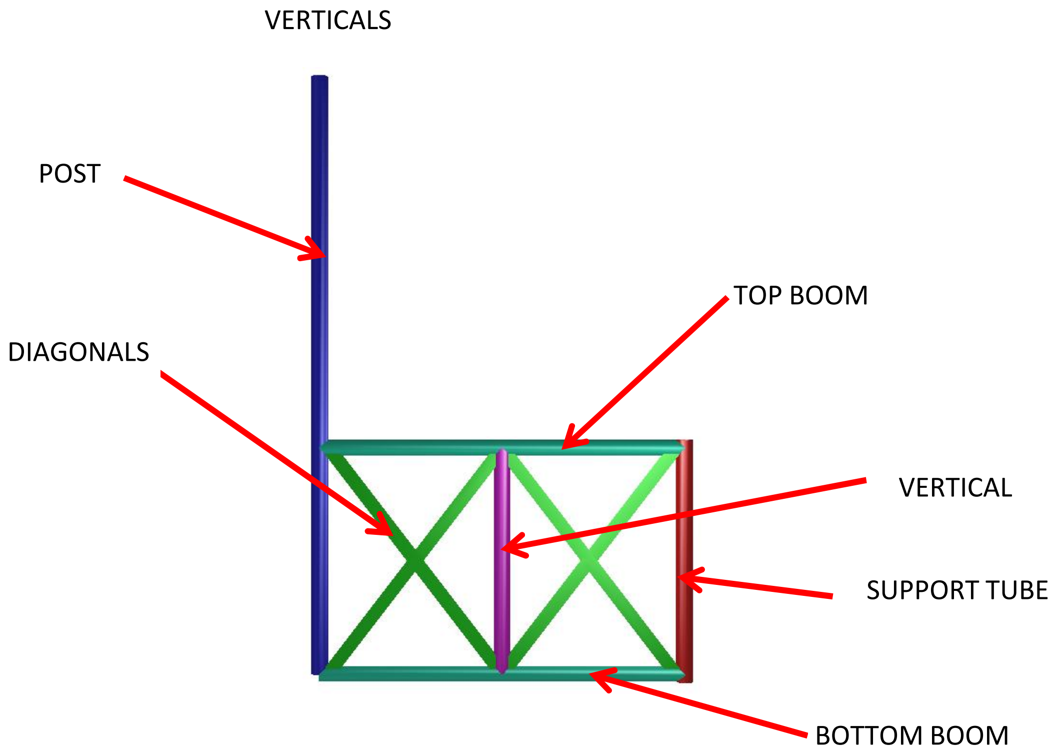
STRAP Model

The structure was analysed in STRAP structural analysis program.



The section sizes are:

Top boom	48.3 x 4.4 CHS
Bottom boom	48.3 x 4.4 CHS
Verticals	48.3 x 4.4 CHS
Diagonals	38x 19 x 3.25 Oval
Post	48.3 x 4.4 CHS
Support Tube	48.3 x 4.4 CHS

Applied loads - UDL

Aload of 15kN/m^2 applied as a UDL

Assuming the frames are at 1.00 metre intervals then the load on the top boom of the frame will be

w_L= 15.00 kN/m

In addition a 2.5% notional load is to be applied horizontally.

H= w_L*L*0.025 w_L= 15.00 kN/m

L= 1.00 m

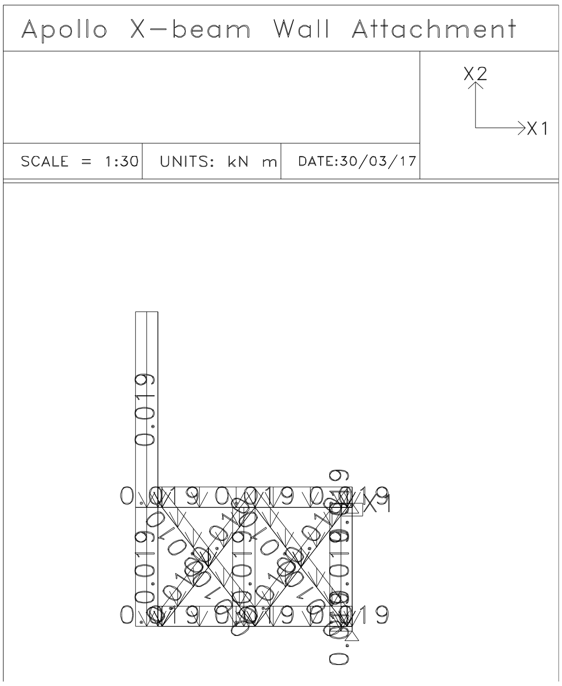
= 15*1*0.025

= 0.38 kN

Load Cases

Load Case 1 **Self Weight**

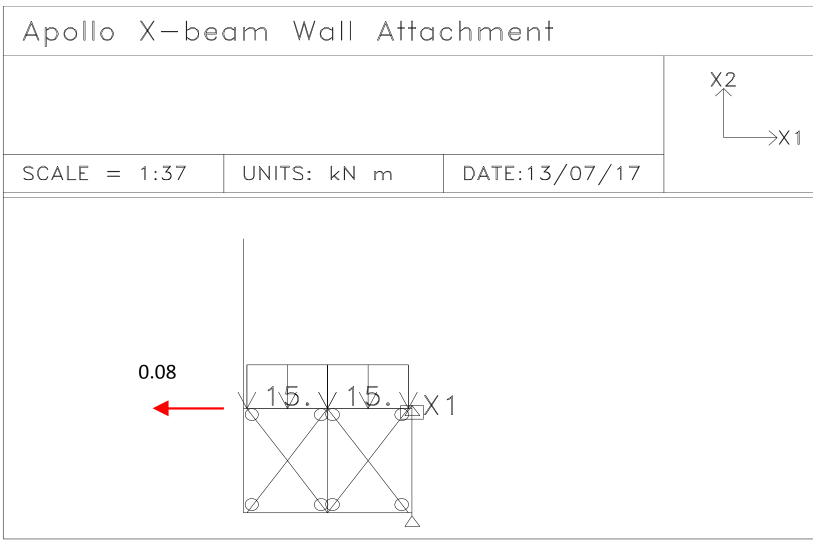
Self weight of all members is factored by 1.15 to account for connections



Load Case 2 **UDL**

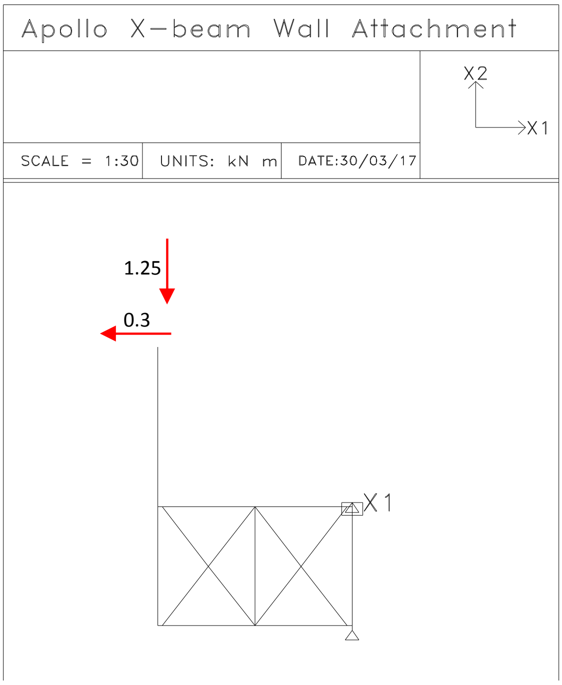
Applied udl line load on top boom, with notional horizontal load

w=	3.00	kN/m (assuming frames at 1m c/c)
H=	0.80	kN



Load Case 3

Handrail loads



Load Combinations

Combination Number	Combination Description	Load Cases
1	ULS	1+2+3
2	SLS	1+2+3

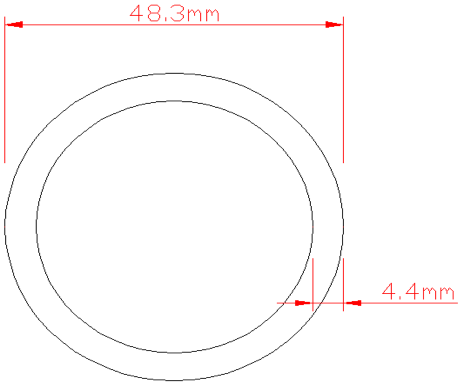
Above Combinations were checked for the following ULS design factors:

$\gamma_D =$ 1.35

$\gamma_L =$ 1.50

Main Boom Layout
ø48.3mm x 4.4mm CHS 6082-T6

Alu. 6082-T2	$P_{o,haz}$ =	0.50		(Table 3.2b)
	$P_{u,haz}$ =	0.64		
	f_o =	250	N/mm^2	
	f_u =	290	N/mm^2	



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

Main Boom 48.3 x 4.4 CHS HAZ

There is a HAZ at welded joint to the vertical and a full welded end at the coupler connection

Take fully section as HAZ affected

$P_{o,haz}$ HAZ Section Layout

Take section shown as fully HAZ using p_{ohaz} of 0.50

A=	303	mm^2
I=	62820	mm^4
W_{el}=	2862	mm^3
W_{pl}=	3829	mm^3
ry=	14.7	mm

$P_{u,haz}$ HAZ Section Layout

Take section shown as fully HAZ using p_{uhaz} of 0.64

A=	374	mm^2
I=	84308	mm^4
W_{el}=	3841	mm^3
W_{pl}=	5059	mm^3
ry=	13.9	mm

Main Boom Moment Capacity (6.2.5.1)

Non-HAZ

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

α = W_{pl}/Wel (Table 6.4)
1.35

W_{el} = 6.11 cm^3

f_o = 250 N/mm^2

γ_{M1} = 1.1 (6.1.3)

= 1.35*6.11*250/1100

$M_{o,Rd}$ = 1.87 kNm

HAZ

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

$W_{u,eff}$ = $W_{el,PuHAZ}$
= 3.84 cm^3

f_u = 290 N/mm^2

γ_{M2} = 1.25 (6.1.3)

= 3.84*290/1250

$M_{u,Rd}$ = 0.89 kNm

$M_{Rd,x}$ = 0.89 kNm   lesser value of $M_{c,Rd}$ / $M_{u,Rd}$

Main Boom Shear Capacity (6.2.6)

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

$A_v =$ 0.6A

$A_v =$ 0.6*303

$A_v =$ 181.80 mm^2

$\gamma_{M1} =$ 1.1

$f_o =$ 250 N/mm^2

= 181.8*250/(SQRT(3)*1100)

$V_{Rd} =$ 23.86 kN

Main Boom Axial Comp Capacity @ 1000mm (effective length of beam between restraints)

$N_{b,Rd} = X_{haz}\omega_{x,haz} A_{u,eff} f_u / \gamma_{M2}$ (6.3.1.1 (6.49b))

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

E = 70,000 N/mm^2

I = 147,654 mm^4

k = 0.50 (Table I.2)

L = 1,000 mm

$N_{cr} =$ ((PI()^2)*70000*147654)/((0.5^2)*(1000^2))

= 408,040.24 N

$\lambda = \sqrt{A_{u,eff} f_u / N_{cr}}$ (6.3.1.2)

= 0.48 $A_{u,eff} =$ 374 mm^2

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

$\Phi = 0.5(1+ά(\lambda-\lambda_o)+\lambda^2)$

ά = 0.20 Table 6.6

$\lambda_o =$ 0.10 Table 6.6

$\Phi =$ 0.66

X = 0.911

$\omega_{x,haz} = 1/(X_{haz}+(1-X_{haz})\sin(PI() x_{s,haz}/lcr))$

= 1.10

$N_{b,Rd} =$ 0.911*1.1*374*290/1250

= 86.95 kN

Main Boom Axial Tension Capacity (6.2.3)

1. General yielding

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

$f_o =$ 250 N/mm^2

$A_g =$ A

$=$ 607 mm^2

$\gamma_{M1} =$ 1.1

= 607*250/1100

= 137.95 kN

2. Local failure

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

$f_u =$ 290 N/mm^2

$A_{u,eff} =$ 374 mm^2

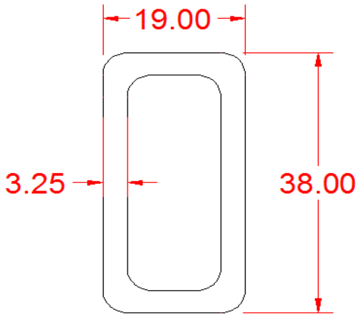
$\gamma_{M1} =$ 1.25

= 374*290/1250

= 86.77 kN

Lesser Value= 86.77 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	$\beta=$	b/t	b= 38-2*3.25
			= 31.50
	=	9.69	t = 3.25
	$\varepsilon=$	$sqrt(250/f_o)$	$f_o=$ 250N/mm^2
	=	1.00	
Class A, without welds, Internal parts	$\beta_1=$	11ε	
	=	11*1.0	
	=	11.00	
	>	9.69	

Section is class 1

Diagonal Axial Comp Capacity @ 400mm (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
$I =$ 53,341 mm^4
$k =$ 0.50
$L =$ 400 mm

$N_{cr} =$ ((PI()^2)*70000*110839)/((0.5^2)*(400^2))
= 1,914,389.89 N

$\lambda = \sqrt{A_{eff} f_o / N_{cr}}$ (6.3.1.2)
= 0.18 $A_{eff} =$ 256 mm^2

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

$\Phi = 0.5(1+ά(\lambda-\lambda_o)+\lambda^2)$
ά = 0.20 Table 6.6
$\lambda_o =$ 0.10 Table 6.6
$\Phi =$ 0.52

X = 0.96

$\omega_{x,haz} = 1/(X_{haz}+(1-X_{haz})\sin(PI()x_{s,haz}/lcr))$

= 1.04

$N_{b,Rd} =$ 0.96*1.04*256*290/1250
= 59.30 kN

Diagonal Axial Tension Capacity (6.2.3)

General yielding

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

$f_o =$ 250 N/mm^2
$A_g =$ 328 mm^2
$\gamma_{M1} =$ 1.1

= 406*250/1100
= 92.27 kN

Local failure

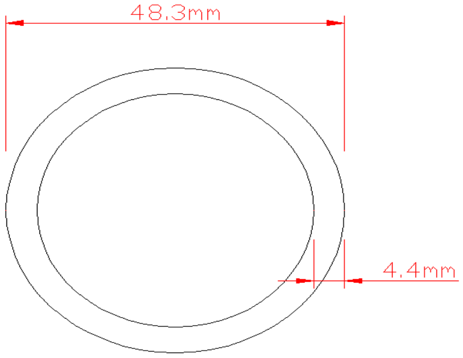
$N_{u,Rd} = A_{net} f_u / \gamma_{M2}$

$f_u =$ 290 N/mm^2
$A_{net} = \rho_{u,haz} * A$
= 0.64*406
= 259.84 mm^2
$\gamma_{M1} =$ 1.25

= 259.84*290/1250
= 60.28 kN

Vertical CHS Member Capacity
ø48.3mm x 4.4mm CHS 6082-T6

Alu. 6082-T2	$P_{o,haz}$ =	0.50	(Table 3.2b)
	$P_{u,haz}$ =	0.64	
	f_o =	250 N/mm^2	
	f_u =	290 N/mm^2	



Section Properties

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

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

Class A, without welds, Internal parts	β_1=	11ε
	=	11*1.0
	=	11.00
	>	10.98

Section is class 1

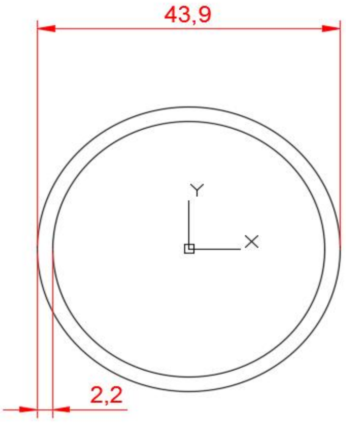
Vertical CHS HAZ Length

Full perimeter weld at the joint, therefore the entire section is affected by HAZ.

As per BS EN 1999-1-1, for HAZ wall thickness factored by 0.50 (For $P_{o,haz}$)

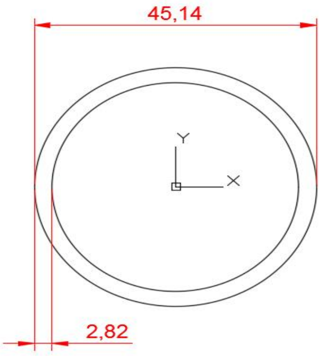
As per BS EN 1999-1-1, for HAZ wall thickness factored by 0.64 (For $P_{u,haz}$)

Vertical CHS $P_{o,haz}$ HAZ Section Layout



A_{haz}=	288	mm^2
I=	62820	mm^4
I_z=	62820	mm^4
$W_{el,PoHAZ}$=	2,862	mm^3
$W_{pl,PoHAZ}$=	3,864	mm^3

Vertical CHS $P_{u,haz}$ HAZ Section Layout



A_{haz}=	374	mm^2
I=	84308	mm^4
I_z=	84308	mm^4
$W_{el,PuHAZ}$=	3,735	mm^3
$W_{pl,PuHAZ}$=	5,043	mm^3

Vertical CHS Moment Capacity (6.2.5.1)

Non-HAZ

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

$\alpha =$ W_{pl}/W_{el} (Table 6.4)

$=$ 1.35

$W_{el} =$ 6.11 cm^3

$f_o =$ 250 N/mm^2

$\gamma_{M1} =$ 1.1 (6.1.3)

= 1.35*6.11*250/1100

$M_{o,Rd} =$ 1.87 kNm

HAZ

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

$W_{net} =$ $W_{el,PuHAZ}$

$=$ 3.74 cm^3

$f_u =$ 290 N/mm^2

$\gamma_{M2} =$ 1.25 (6.1.3)

= 3.74*290/1250

$M_{u,Rd} =$ 0.87 kNm

$M_{Rd,x} =$ 0.87 kNm   lesser value of $M_{c,Rd}$ / $M_{u,Rd}$

Vertical CHS Shear Capacity (6.2.6)

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

$A_v =$ $0.6A_e$

$A_v =$ 0.6*288

$A_v =$ 172.80 mm^2

$\gamma_{M1} =$ 1.1

$f_o =$ 250 N/mm^2

= 172.80*250/(SQRT(3)*1100)

$V_{Rd} =$ 22.67 kN

Vertical CHS Axial Comp Capacity @ 650mm (effective length of beam)

Localised Weld $N_{b,Rd} = X_{haz}\omega_{x,haz} A_{u,eff} f_u / \gamma_{M2}$ (6.3.1.1 (6.49b))

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

$E =$ 70,000 N/mm^2

$I =$ 147,654 mm^4

$k =$ 0.50

$L =$ 650 mm

$N_{cr} =$ ((PI()^2)*70000*147654)/((0.5^2)*(650^2))

= 965,775.71 N

$\lambda_{haz} = \sqrt{A_{u,eff} f_u / N_{cr}}$ (6.3.1.2)

= 0.31

$A_{u,eff} =$ 374 mm^2

$A =$ 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

$\Phi =$ 0.57

$X =$ 0.88

$\omega_{x,haz} = 1/(X_{haz}+(1-X_{haz})\sin(PI()x_{s,haz}/lcr))$

= 1.12

$N_{b,Rd} =$ 0.88*1.12*374*290/1250

= 85.52 kN

Vertical CHS Axial Tension Capacity (6.2.3)

1. General yielding

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

$f_o =$ 250 N/mm^2

$A_g =$ A

$=$ 607 mm^2

$\gamma_{M1} =$ 1.1

= 607*250/1100

= 137.95 kN

2. Local failure

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

$f_u =$ 290 N/mm^2

$A_{u,eff} =$ 374 mm^2

$\gamma_{M1} =$ 1.25

= 374*290/1250

= 86.77 kN

Lesser Value= 86.77 kN

Posts

The Handrail post and the support tube are both unwelded steel Scaffold tubes
The capacity of these are as shown below

Where safe working values are taken from TG20 , a partial factor of 1.5
is applied in accordance with the notes to table 5.9

Moment capacity

$M_R=$ Sfy/γ_M From TG20

$S=$ 7.87 cm^3

$f_y=$ 235 N/mm^2

$\gamma_M=$ 1.1

= 7.87*235/1100

= 1.68 kNm

Shear Capacity

$V_R=$ **43.65 kN** From TG20

(Ultimate value = Safe*1.5)

Compression capacity

For a length of 1.1m, (handrail post) the compression capacity is

$N_{Ec}=$ **73.35 kN** From TG20

(Ultimate value = Safe*1.5)

Tensile capacity

$N_{Et}=$ Afy/γ_M From TG20

$A=$ 5.57 cm^2

$f_y=$ 235 N/mm^2

$\gamma_M=$ 1.1

= 557*235/1100

= **119.00 kN**

CALCULATION SHEET	Project :	Apollo 1.0m X-beam Wall Attachment			ALAN WHITE DESIGN
	Element :	Results - UDL			
	Job Number :	X0069	By : anw	Date:Mar 17	
	Document No :	001A	Checked:mmr	Date:Mar 17	

Results	Action	Formula	Ultimate	Calculated	Factor
Boom Capacity					
	Moment	$M_{Rd,x}$	0.89	0.34	2.66
	Shear	V_{Rd}	23.86	12.60	1.89
	Tension	$N_{u,Rd}$	30.00	19.50	1.54
	Compression	$N_{b,Rd}$	86.95	19.05	4.56
	Combined	< 1.0	1.00	0.95	1.06
Diagonal Capacity					
	Tension	$N_{u,Rd}$	60.28	8.29	7.27
	Compression	$N_{b,Rd}$	59.30	14.85	3.99
Vertical Capacity					
	Moment	$M_{Rd,x}$	0.87	0.05	17.35
	Shear	V_{Rd}	22.67	0.10	226.74
	Tension	$N_{u,Rd}$	86.77	0.00	867680.00
	Compression	$N_{b,Rd}$	85.52	0.00	85518.54
	Combined	< 1.0	1.00	0.06	17.35
Handrail post capacity					
	Moment	$M_{Rd,x}$	1.68	0.52	3.23
	Shear	V_{Rd}	43.65	0.45	97.00
	Tension	$N_{u,Rd}$	119.00	0.00	118995.45
	Compression	$N_{b,Rd}$	73.35	2.82	26.01
	Combined	< 1.0	1.00	0.32	3.09
Support tube capacity					
	Moment	$M_{Rd,x}$	1.68	0.49	3.43
	Shear	V_{Rd}	43.65	19.70	2.22
	Tension	$N_{u,Rd}$	119.00	0.00	118995.45
	Compression	$N_{b,Rd}$	73.35	24.56	2.99
	Combined	< 1.0	1.00	0.53	1.88
				minimum factor is	1.06

Take max values as 15kN/m2

CALCULATION SHEET	Project :	Apollo 1.0m X-beam Wall Attachment			ALAN WHITE DESIGN
	Element :	Results -Point load of 10kN			
	Job Number :	X0069	By : anw	Date:Mar 17	
	Document No :	001A	Checked:mmr	Date:Mar 17	

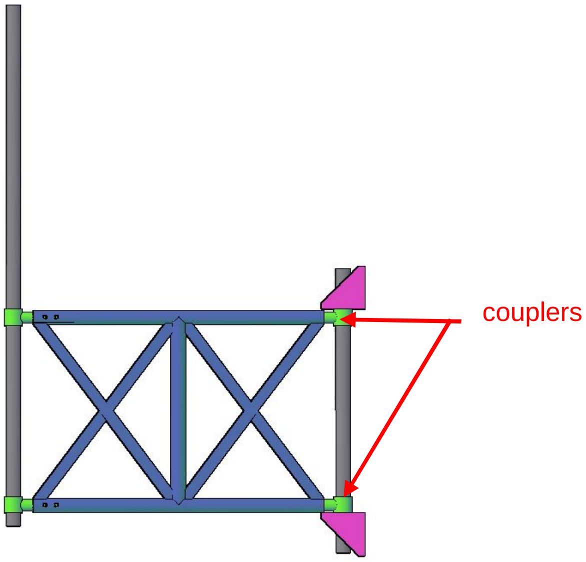
Results	Action	Formula	Ultimate	Calculated	Factor
Boom Capacity					
	Moment	$M_{Rd,x}$	0.89	0.43	2.07
	Shear	V_{Rd}	23.86	12.60	1.89
	Tension	$N_{u,Rd}$	30.00	25.47	1.18
	Compression	$N_{b,Rd}$	86.95	24.47	3.55
	Combined	< 1.0	1.00	1.29	0.77
Diagonal Capacity					
	Tension	$N_{u,Rd}$	60.28	8.97	6.72
	Compression	$N_{b,Rd}$	59.30	11.52	5.15
Vertical Capacity					
	Moment	$M_{Rd,x}$	0.87	0.07	12.40
	Shear	V_{Rd}	22.67	0.20	113.37
	Tension	$N_{u,Rd}$	86.77	0.00	867680.00
	Compression	$N_{b,Rd}$	85.52	1.44	59.39
	Combined	< 1.0	1.00	0.09	11.68
Handrail post capacity					
	Moment	$M_{Rd,x}$	1.68	0.52	3.23
	Shear	V_{Rd}	43.65	0.45	97.00
	Tension	$N_{u,Rd}$	119.00	0.00	118995.45
	Compression	$N_{b,Rd}$	73.35	2.82	26.01
	Combined	< 1.0	1.00	0.32	3.09
Support tube capacity					
	Moment	$M_{Rd,x}$	1.68	0.62	2.71
	Shear	V_{Rd}	43.65	24.95	1.75
	Tension	$N_{u,Rd}$	119.00	0.00	118995.45
	Compression	$N_{b,Rd}$	73.35	17.10	4.29
	Combined	< 1.0	1.00	0.52	1.93
				minimum factor is	0.77

So maxpoint load is

P= 10 * factor
= 10*0.77
= 7.70 kN
= 7.50 kN say

Boom end connection

At each end of both booms of the X-beam , a half coupler is connected



A half coupler is bolted to an aluminium plug which is welded into the end of the X-beam boom

The nut on the coupler bolt is welded to the bolt to prevent it from turning and so the capacity of this component relies on

1 Capacity of the coupler
2 Capacity of bolt
3 Capacity of the fillet weld

Coupler capacity

From BS EN 12811-1, Table C.1, Tensile capacity of the coupler is

N_{Rt}= **30 kN**

Bolt capacity

For M12 grade 8.8 bolt

N_{Rt}= **34 kN**

Weld capacity

throat width is

a= 0.7s   s= 5 mm
= 0.7*5
= 3.50 mm

length of weld

l= PI()*(48.3-4.4*2) -2a
PI()*(48.3-4.4*2) -2*3.5
= 117.09 mm

A_w= l.a
= 117.09*3.5
= 409.82 mm^2

$\sigma_\perp$= f_w/γ_{Mw} f_w= 190 N/mm^2
γ_{Mw}= **1.25**
= 190/1.25
= 152.00 N/mm^2

so capacity

N_{Rt}= $\sigma_\perp * A_w$
= 152*409.82/1000
= **62.29 kN**

Capacity

From above the limiting capacity is the half coupler

so N_{Rt}= **30 kN**

From strap analysis the max tension in the reactions is

N_E= **19.50 kN**
< **30.00 kN**   **ok**

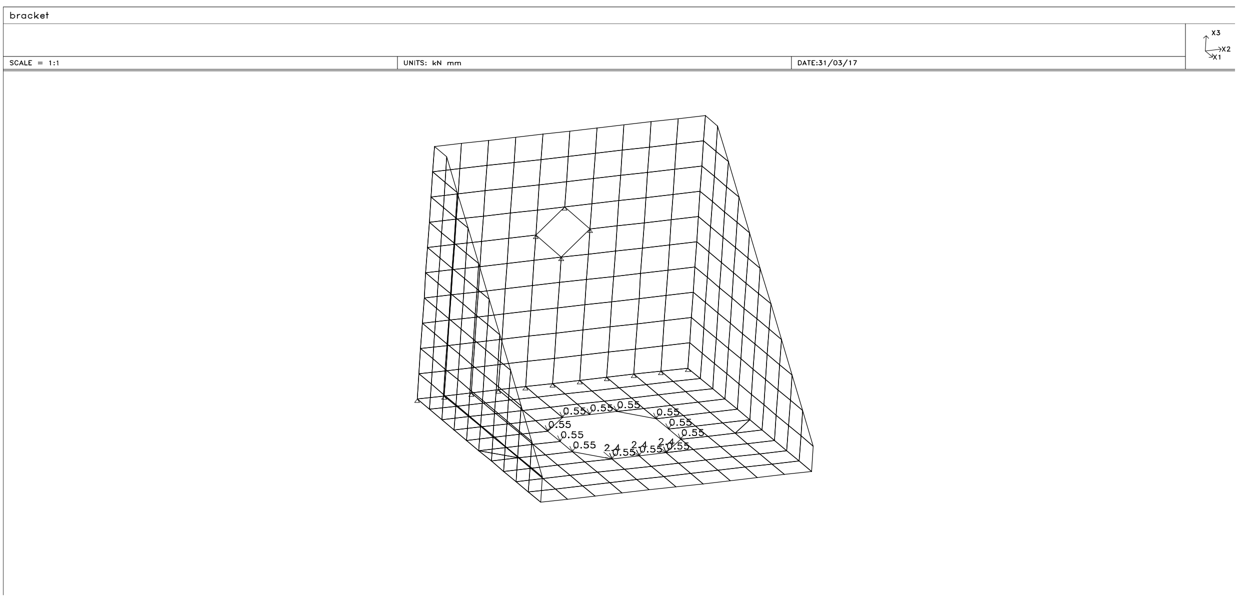
Bracket

The bracket used to connect the support tube to the wall is fabricated from mild steel plate.

This is modelled in STRAP as shown above, allowing for the voids for the scaffold tube and the bolt

Loading

The reaction loads from the analysis of the frame were applied as shown

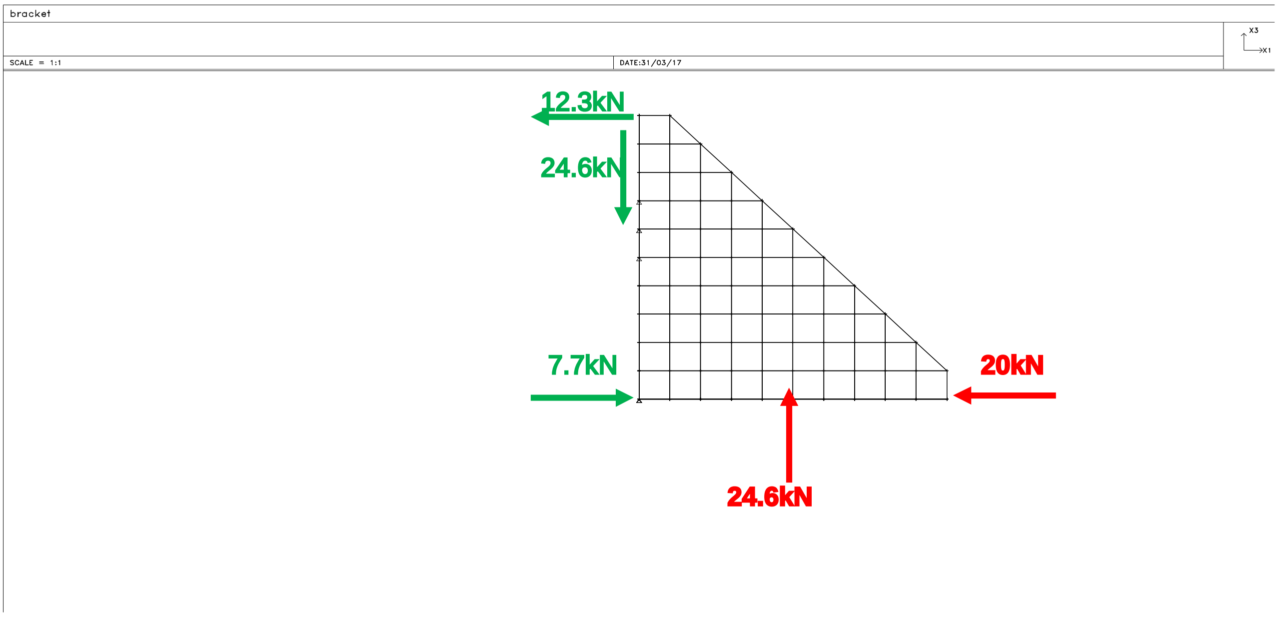


The vertical load of 6.6kN was applied at twelve nodes at 0.55kN per node

The horizontal load of 7.2kN was applied to three nodes at 2.4kN per node

Supports - compression and vertical

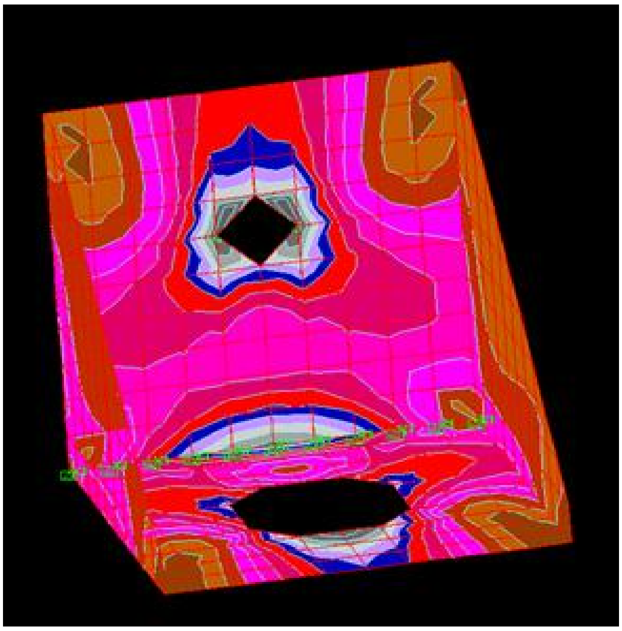
The bracket was supported at the bolt hole and along the bottom edge as shown



The support reactions in the bolt are

Tension in bolt	**0.00**	**kN unfactored**
Shear in bolt	**24.50**	**kN unfactored**

Results



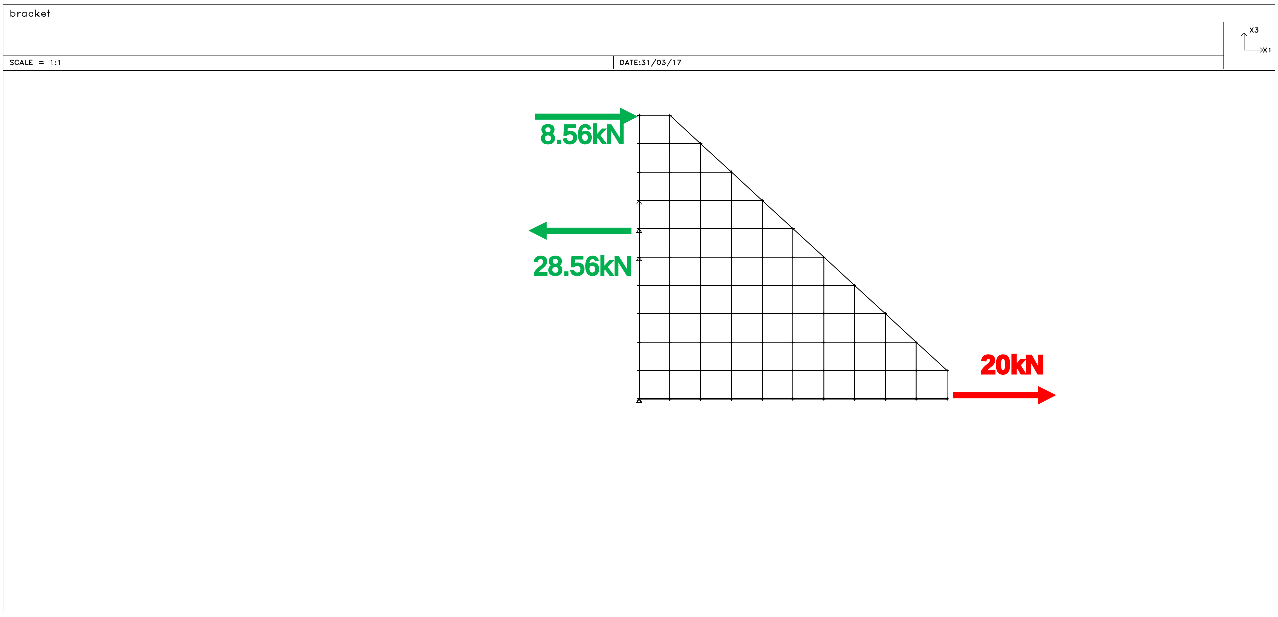
The maximum results gave a stress in the bracket steel of

f=	**192.00 N/mm^2**	
=	**275.00 N/mm^2**	**ok**

CALCULATION SHEET	Project :	Apollo 1.0m X-beam Wall Attachment			ALAN WHITE DESIGN
	Element :	Bracket			
	Job Number :	X0069	By : anw	Date:Mar 17	
	Document No :	001A	Checked:mmr	Date:Mar 17	

Supports - tensile

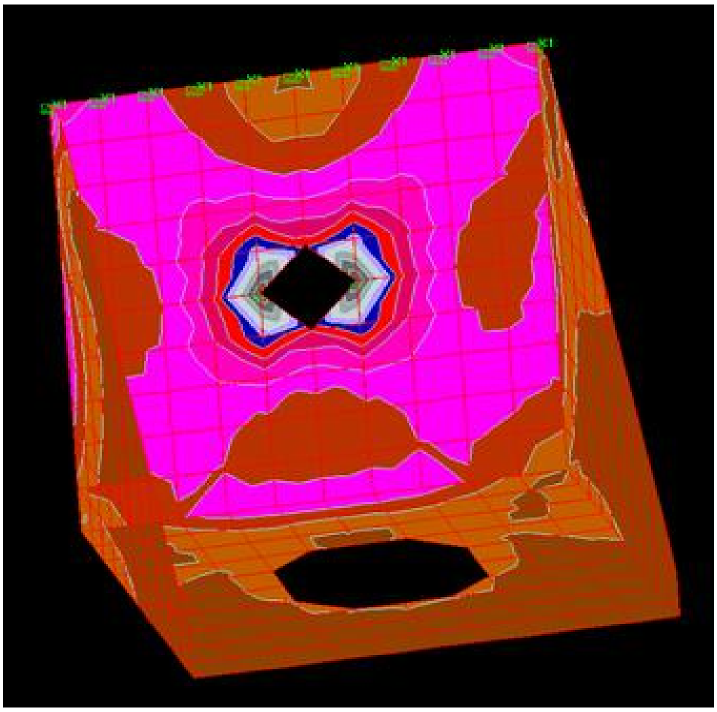
The bracket was supported at the bolt hole and along the top edge as shown



The support reactions in the bolt are

Tension in bolt **20.00 kN unfactored**

Results



The element results gave a stress in the bracket steel of

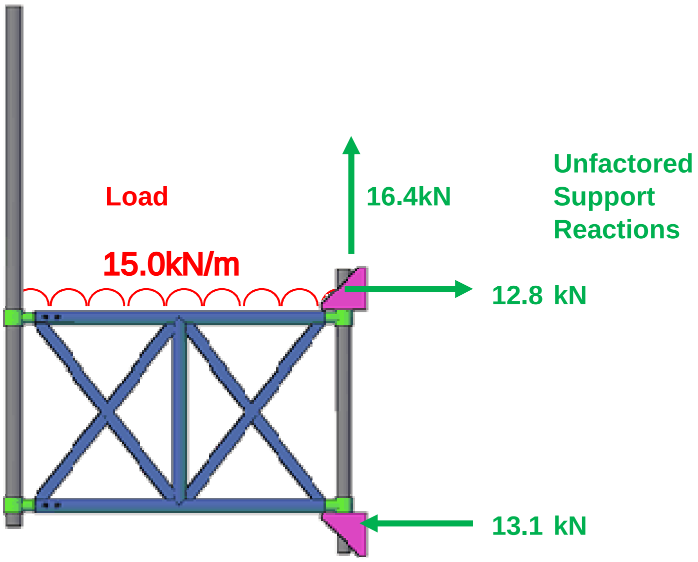
f= **269.00 N/mm^2**

= **275.00 N/mm^2** **ok**

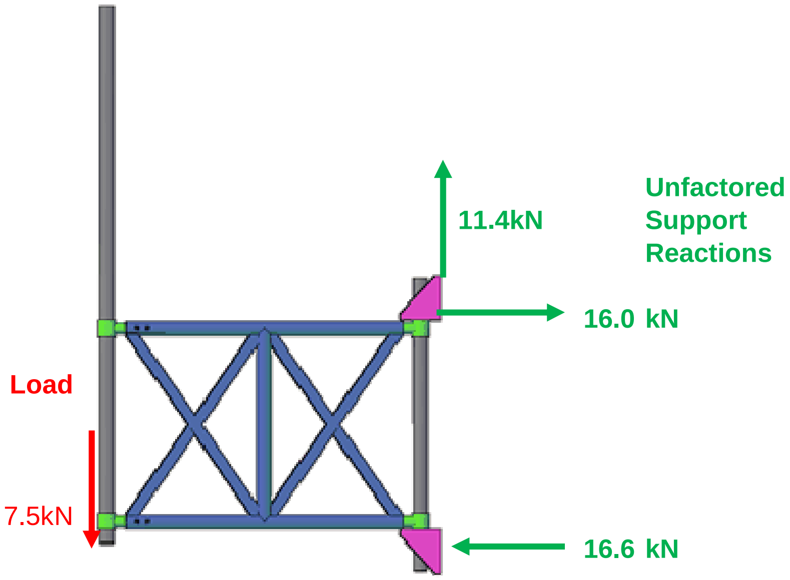
Summary

The X-beam wall attachment has been checked and all components found to be satisfactory for the UDL of 15kN/m or a point load of 7.5kN assuming that the frames are spaced at 1.0m centres

Uniformly Distributed Load



A total allowable load of 15kN or 1500kg uniformly distributed across top boom.



A total allowable point load at end of 7.5kN or 750kg