Engineering That Brings Design To Life.

Structural Calculations

CO21/371/09

Vertical Planters

Huddersfield

Revision	Date	Changes
-	20.10.2021	Initial Issue
A	26.10.2021	Pad foundation size revised
A(1)	28.10.2021	Baseplate thickness revised

General Construction Notes and Guidance on using these Calculations

1. Calculations are not to be used for the purpose of ordering materials and should only be used for Building Regulations submissions. All dimensions should be checked by the contractor on site.

2. All steelwork to be mechanically wire brushed and painted two coats of red oxide. Steelwork located in the cavity or below DPC to be suitably protected with 2 coats of bituminous paint.

3. All steelwork connections to use grade 8.8 bolts unless stated otherwise. These are to be spanner tightened using the appropriate podger spanner (min length 460mm) or suitable power tools in accordance with BS2583. If a torque wrench is used the torque applied should be around 90Nm for M16 bolts, 110Nm for M20 & 130Nm for M24.

4. All timber to be grade C24 (SC4), unless stated otherwise. Preservative treated to Architects details.

5. To be read in conjunction with Architects drawings, any inconsistencies between the drawings should be reported. If any site conditions or existing details are found that may affect the structural design, JMS Consulting Engineers are to be notified immediately.

6. For details of fire protection to steelwork, see Architects drawings.-371

7. The Contractor is to ensure that all existing construction is adequately supported, using needles and props as required. Where a new beam supports the existing construction, adequate pre-load is to be applied and suitable packs such as driven dry-slate introduced, then pointed up with mortar.

8. All blockwork to be 7.3 N/mm^2 in class III mortar below DPC in accordance with BS 5628 : Part 3 : 2005 or suitable 7.0 N/mm^2 foundation quality blocks in class II mortar in accordance with the manufacturer's instructions. All brickwork below DPC to be Engineering Bricks DPC in accordance with BS 5628 : Part 3 : 2005.

9. The project requires the introduction of heavy structural elements such as steel beams or concrete lintels. Although the Construction (Design and Management) Regulation 2015 would not normally apply to this type of construction, the designer still has an obligation to foresee risks and bring to the attention of the builder such risks. In consequence, the builder is to take into consideration the placement of all structural elements, ensuring that the method of lifting and placement is safely carried out. Responsibility for this element lies with the Contractor. As the existing walls need to be propped in order to introduce some of the lintels, this should also be considered in relationship to the risk assessment of the Contractor. Safe working procedures must be adopted. Responsibility for this element lies with the Contractor. Splice details for long-span beams can often be accommodated if required.

10. All construction products should be CE marked in accordance current legislation. This includes all fabricated structural steelwork in accordance with BS EN 1090-1 and BS EN 1090-2. The consequence class is CC2 unless noted otherwise. The service class is SC1 for all buildings, SC2 for all lifting beams, sculptures & fall arrest systems. Production category will be PC1 unless noted otherwise. All site welded items, S355 steelwork & CHS lattice girders will be PC2. As such the execution class for buildings will be EXC2.

11. CLIMATE CHANGE: The Building Research establishment have produced a document CBG 63 "Climate Change: impact on building design and construction". Part of their recommendations are that designers and builders should give consideration to:
    a. Increased wind loading by providing additional laps and fixings to roof coverings

    b. Consider foundation depth on shrinkable clays and to avoid future problems, increase the depth above standard requirements if there is a risk. This should be in accordance with the NHBC Standards, Chapter 4.2 Guidance on Building near Trees. If the calculations do not specifically design the depths of the foundations to take into account any local trees, then this should be checked and agreed with the Building Inspector on site.

Party Wall etc. Act 1996

If part of the work is adjacent to the boundary, the adjacent neighbours right to support could be affected; the issues associated with Party Wall Act may need to be considered. This may include providing information to the adjoining owner, giving sufficient notice of works in compliance with the Act. If the following list applies to this project then the Party Wall Act will apply.

- Installing a new beam into the shared wall between properties
- Demolishing, building or under-pinning an existing shared wall
- Building a new wall at or on the boundary or junction of two properties
- Damp-proofing all the way through a party wall
- Digging foundations that are within 3m of a Party Wall, where the new foundations are deeper than the existing ones
- Where the new foundations are within 6m and lower than a 45° line from the bottom of the existing foundations

Wind load on sculpture

Location:	Huddersfield
Basic Wind Speed (Vb)	= 23 m/s
Site Altitude	= 92m
Nearest distance to sea	= 80 km
Altitude Factor (Sa)	= 1.009

Wind load calculation

H = 12m

Vb = 23 m/s

Vs = Vb . Sa . Sd . Ss . Sp

= 23 x 1.009 x 1.00 x 1.0 x 1.0 = 23.21

Sb = 1.75 Terrain and Building Factor – site in town

Ve = Vs . Sb

= 23.21 x 1.75

= 40.62

qs = 0.613 . Ve^2

= 0.613 x 40.62^2 / 1000

= 1.01 kN/m^2 Dynamic Wind Pressure

Net pressure:

P = qs x Cp x Ca

= 1.01 x 2.0 x 1.0

= 2.02 kN/m^2

Assume 50% porosity

Applied loading = 2.02 * 0.5 **= 1.01kN/m2**

Overturning moment:

Area of top element – consider as trapezium (conservative)

(3m+0.8m)/2 x 6m = 11.4m2

Conservatively assume centroid = 9m

Area of bottom element = 0.8m*6m = 4.8m2

Centroid = 3m

M = (1.01kN/m2 x 11.4m2 x 11m) + (1.01kN/m2 x 4.8m2 x 5m)

= **150 kNm**

Restoring Moment:

SW Sculpture: = 10kN x 1.5m = 15 kNm

Foundation: = (3.0m x 3.0m x 1.25m x 24kN/m3) – (0.7^2m x pi/4*24*1.25) * 1.5m = 390 kNm

= 405 kNm

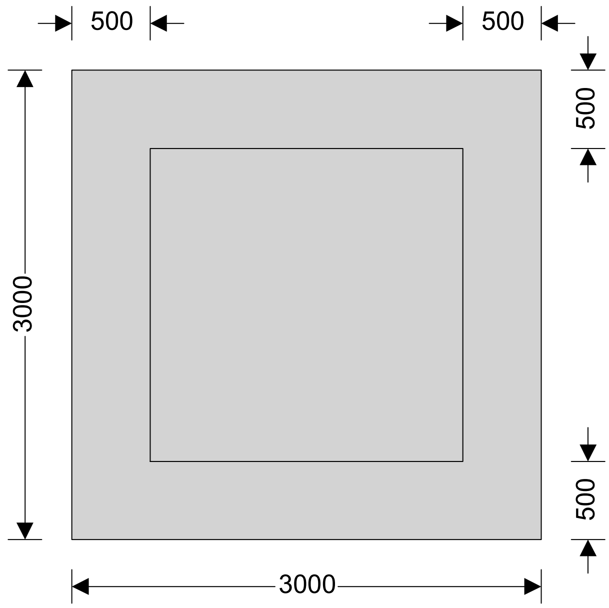
FOS: 405/150 = 2.7 – **>2 Therefore satisfactory**

Base to have 700mm diameter void for planting.

Applied Bearing pressure: - Assume 75kN/m^2 allowable – to be confirmed on site.

PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07



Pad footing details

Length of pad footing L = **3000** mm | Width of pad footing B = **3000** mm

Depth of pad footing h = **1250** mm | Depth of soil over pad footing h_{soil} = **200** mm

Density of concrete ρ_{conc} = **23.6** kN/m^3

Column details

Column base length l_A = **2000** mm | Column base width b_A = **2000** mm

Column eccentricity in x e_{PxA} = **0** mm | Column eccentricity in y e_{PyA} = **0** mm

Soil details

Depth of soil over pad footing h_{soil} = **200** mm | Density of soil ρ_{soil} = **20.0** kN/m^3

Allowable bearing pressure $P_{bearing}$ = **125** kN/m^2

Axial loading on column

Dead axial load P_{GA} = **10.0** kN | Imposed axial load P_{QA} = **0.0** kN

Wind axial load P_{WA} = **0.0** kN | Total axial load P_A = **10.0** kN

Foundation loads

Dead surcharge load F_{Gsur} = **0.000** kN/m^2 | Imposed surcharge load F_{Qsur} = **0.000** kN/m^2

Pad footing self weight F_{swt} = **29.500** kN/m^2

Soil self weight F_{soil} = **4.000** kN/m^2 | Total foundation load F = **301.5** kN

Moment on column base

Dead moment in x direction M_{GxA} = **0.000** kNm | Dead moment in y direction M_{GyA} = **0.000** kNm

Imposed moment in x direction M_{QxA} = **0.000** kNm | Imposed moment in y direction M_{QyA} = **0.000** kNm

Wind moment in x direction M_{WxA} = **0.000** kNm | Wind moment in y direction M_{WyA} = **150.000** kNm

Total moment in x direction M_{xA} = **0.000** kNm   Total moment in y direction M_{yA} = **150.000** kNm

Check stability against overturning in y direction

Total overturning moment M_{yOT} = **150.000** kNm   Total restoring moment M_{yres} = **467.250** kNm

PASS - Restoring moment is greater than overturning moment in y direction

Calculate pad base reaction

Total base reaction T = **311.5** kN

Base reaction eccentricity in x e_{Tx} = **0** mm   Base reaction eccentricity in y e_{Ty} = **482** mm

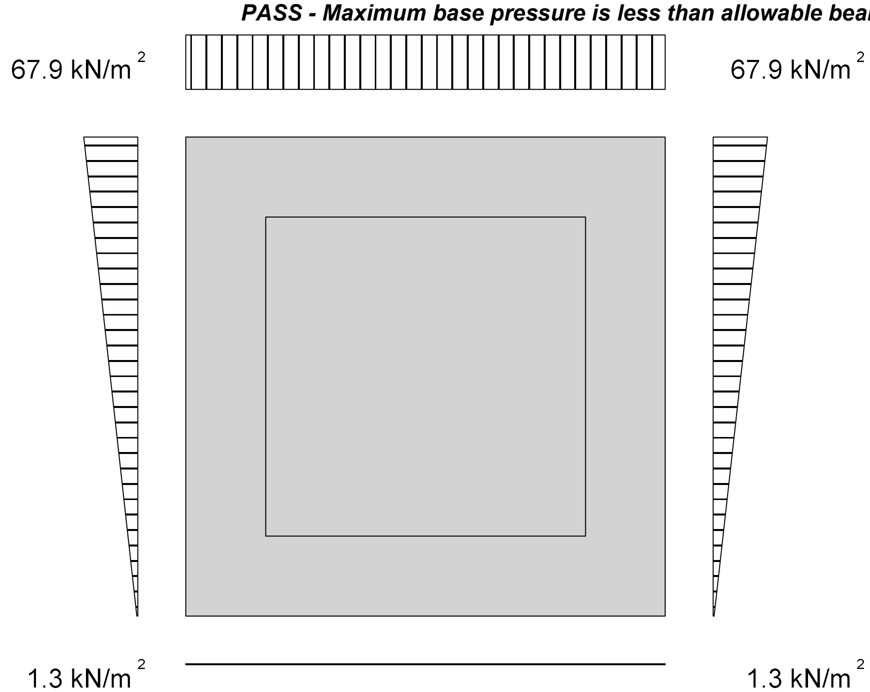
Base reaction acts within middle third of base

Calculate pad base pressures

q_1 = **1.278** kN/m² q_2 = **67.944** kN/m² q_3 = **1.278** kN/m² q_4 = **67.944** kN/m²

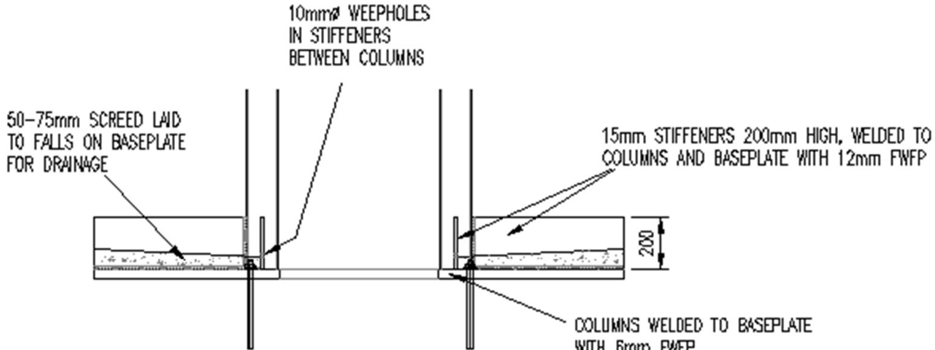
Minimum base pressure q_{min} = **1.278** kN/m²   Maximum base pressure q_{max} = **67.944** kN/m²

PASS - Maximum base pressure is less than allowable bearing pressure



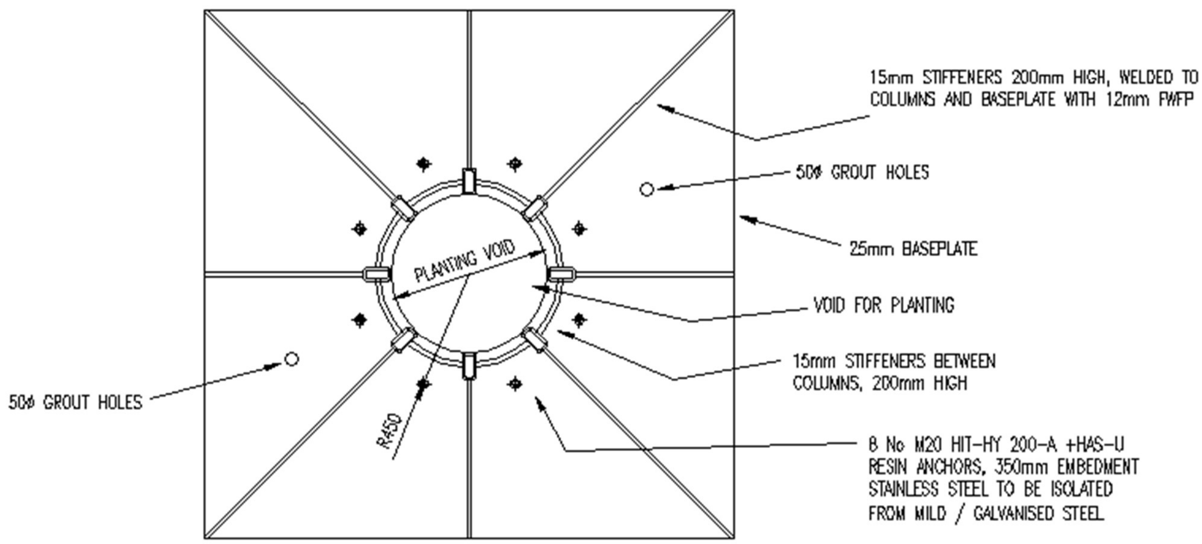
Provide 3.0m x3.0m x1.25m thick mass concrete pad foundation

Baseplate Design:



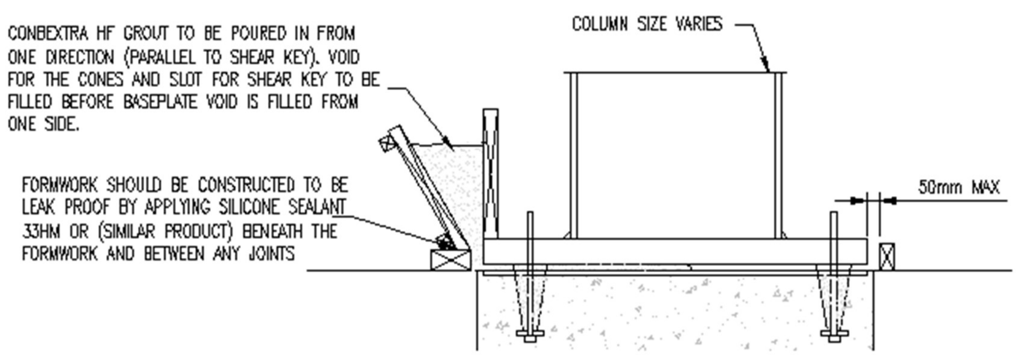
BASEPLATE ELEVATION

SCALE 1:20



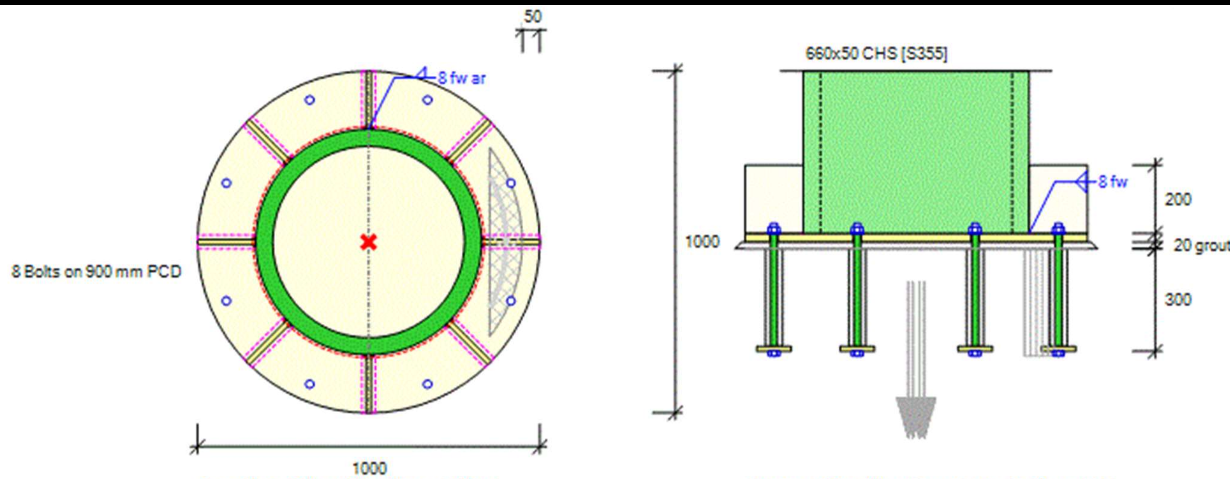
BASEPLATE DETAIL

SCALE 1:20



TYPICAL GROUTING DETAIL FOR LARGE BASEPLATES

SCALE 1:20



Base-Plate 1000 x 1000 x 25 mm (196 kg)
With 8 No. 24 mm holes
For 20 mm Grade 8.8 Bolts.

Washers 100 x 100 x 15 mm thick with 50 mm Void
All Plates S 275
15 mm stiffeners.

Base-Plate Connection to EC 3 (UK NAD)

LOADING CASE 001

Basic Data

User Defined Applied Forces at Interface

Resultant Forces M, Fv, F	Moment +170.0 kNm, Shear +25.0 kN, Axial +15.0 kN (Left side in tension, Axial Compression)		

Basic Dimensions

Column: 660x50CHS [36]	Ø=660.0, T=50.0, py=335		
Bolts 20 mm Ø in 24 mm holes	Grade 8.8 Bolts		
Plates S 275	All weld grades provided to suit minimum connected steel grade		
Grout Fck, Conc Fck, vc, fy, slope	12 N/mm², 20 N/mm², 0.35 N/mm², 265 N/mm², 30 deg to vertical		
Design to	EC 3: Part 1-8: 2005 Design of Connections		
Column Capacities Mc, Fvc, Fc	6246.6 kN.m, 11119.5 kN, 32099.2 kN	Mc = 6246.6 kN.m	OK

Summary of Results (Unity Ratios)

Bolt Tension		0.34	OK
Base-Plate thickness in Compression		1.00	OK
Comp-stiff extending from CHS	0.41, 0.45, 0.34, 0.62, 0.19, 0.74, 0.07, 0.21	0.74	OK
Ten-stiff extending from CHS	0.04, 0.07, 0.05, 0.04, 0.04, 0.06, 0.01, 0.00	0.07	OK
Concrete Embedment		0.64	OK
Washer size	1.00, 1.00	1.00	OK
Pullout Cones	0.47, 0.93	0.93	OK
Horizontal Shear		0.36	OK
Weld	0.02, 0.25, 0.06	0.25	OK

Flange & Web Welds to RHS columns

Load dispersal	Flanges resist Moment and Axial, Web resists Axial and Shear. Direct Bearing therefore design for tensile forces only.		
Areas A, Af, Aw	19.0, 2 x 3.8, 11.5 cm²		

Flange Welds

Fapp=M/la-F•Af/A	2.9/(120.0 - 0.0) - -70.0•3.8/19.0	38.3 kN	
Fapp w=Fapp/B	38.3/(80.0)	0.48 kN/mm	
Weld f_{vw}.d =f_u / 1.73 / (~B_w • γ_{M2})	360/ 1.73 / 0.8 / 1.25	208 N/mm²	
Fcap w=k•1•0.7•f_w •f_{vw}.d	1.225•1•0.7•6•208	1.07 kN/mm	OK

Web Welds

Web weld load=Fv/(2•D)	0.8/(2•120.0)	0.00 kN/mm	
Weld f_{vw}.d=f_u / 1.73 / (~B_w • γ_{M2})	360/ 1.73 / 0.8 / 1.25	208 N/mm²	
Fcap w=1•0.7•f_w •f_{vw}.d	1•0.7•6•208	0.87 kN/mm	OK
F_{axial}=F•Aw/A	70.0•11.5/19.0	42.4 kN	
Web weld load=Ft/(2•D)	42.4/(2•120.0) (on 1 weld/mm)	0.18 kN/mm	
Weld f_{vw}.d=f_u / 1.73 / (~B_w • γ_{M2})	360/ 1.73 / 0.8 / 1.25	208 N/mm²	
Fcap w=1•k•0.7•f_w •f_{vw}.d	1•1.225•0.7•6•208	1.07 kN/mm	OK
σ_{perp}, τ_{perp}	0.18 • 0.707 / a • 1000	29.42 N/mm²	
τ_{parl}	0.00 / a • 1000	0.78 N/mm²	

$f_{w.Ed}=\sqrt{(\sigma_{perp}{}^2+3\bullet(\tau_{perp}{}^2+\tau_{parl}{}^2))}$	$\sqrt{(29.4^2 + 3\bullet(29.4^2 + 0.8^2))}$	58.9 N/mm²	cl 4.5.3.2
$f_{w.Rd}=f_u /(\sim B_w \bullet \gamma_{M2})$	360.0 / (0.8 • 1.25)	360.0 N/mm²	OK
and $\sigma_{perp} <= 0.9\bullet f_u / \gamma_{M2}$	29.4 <= 0.9 • 360 / 1.25	259.2 N/mm²	OK
	No Shear or Resultant Axial Force in Web.		

Maximum pull out force per bolt – 46kN

Resin anchor designed using Hilti Profis design software.

2 Proof I Utilisation (Governing Cases)

		Design values [kN]		Utilization	
Loading	**Proof**	**Load**	**Capacity**	**β_N / β_V [%]**	**Status**
Tension	Concrete Breakout failure	48.000	51.851	93 / -	OK
Shear	-	-	-	- / -	N/A

Loading	β_N	β_V	α	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	-	-	-	-	N/A

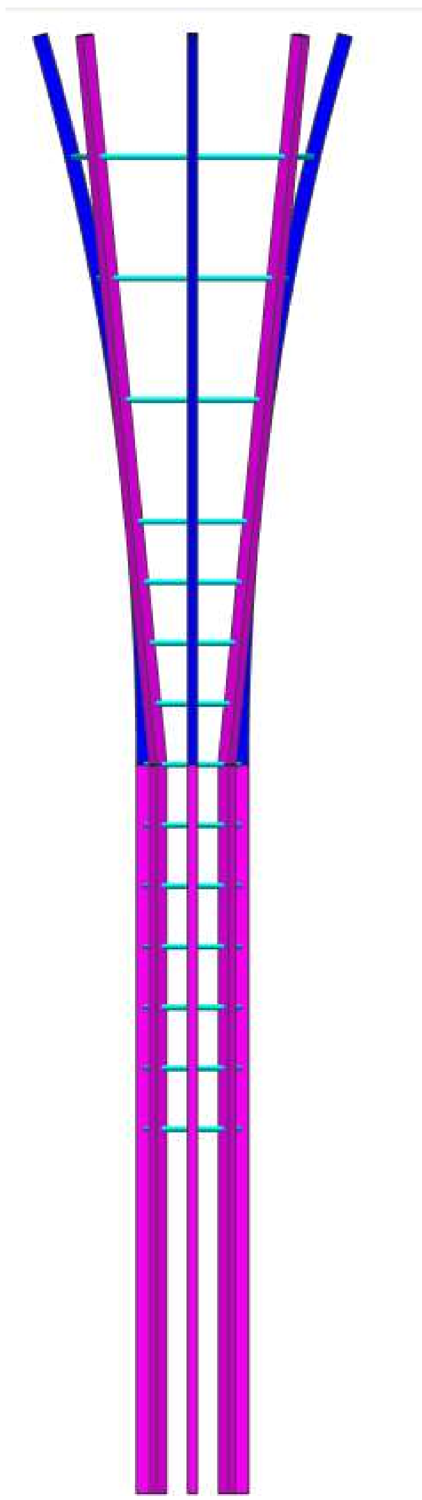
3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

Provide :- **HIT-HY 200-A + HAS-U 5.8 M20 – 300mm embedment**

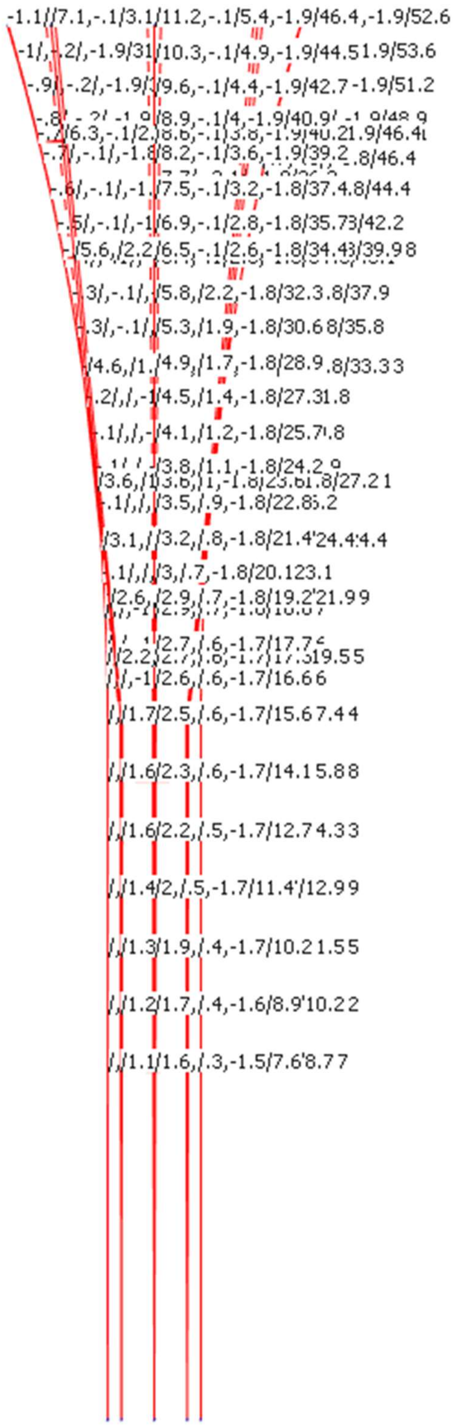
Structural Model

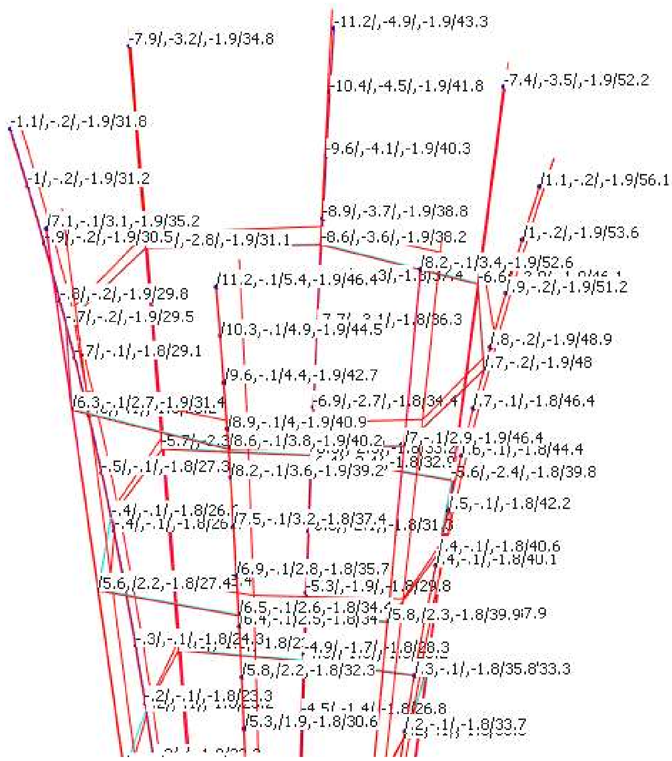


Section Size
Default Grade S 355
48.3x5 CHS 5.34
120x80x5 RHS 14.7 [S 235]
120x80x5 RHS 14.7 [S 235]

Frame to be fully welded together

Deflections in mm (SLS)



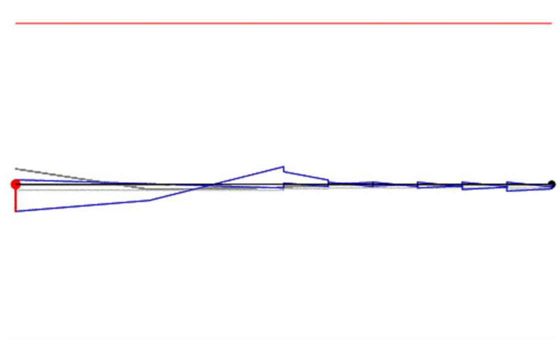


AXIAL WITH MOMENTS (MEMBER)
Member SCL1Id 2 @ Level 1 in Load Case 3 : Columns

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D0 + 1.35 D1 + 1.5 L1

+ 0.75 W1

D1 D 077.010 (kN/m³)
L1 PN -001.100 1.500 (kN,m)



Member Forces in Load Case 3 and Maximum Deflection from Load Case 4										
Member No.	Node End 1 End 2	Axial Force (kN)	Torque Moment (kNm)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm @ m)		Max Def (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
	14	73.91C	0.63	-0.80	0.30	2.94	-0.50	-1.85	0.38	1.48
	63	31.37C	-0.39	-0.47	1.04	0.49	0.28	@ 2.969	@ 2.969	@ 1.224

Classification and Effective Area (EN 1993: 2006)

Section (14.7 kg/m) 120x80x5 RHS 14.7 [S 235]
Class = Fn(b/t,d/t,f_y,N,M_y,M_z) 13, 21, 235, 73.909, 2.942, 0.504 (Axial: Non-Slender) Class 1

Auto Design Load Cases 3, 5, 7, 9, 11, 13, 15 & 17

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.798 / 152.474 =	0.005	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	235 x 74.59/1	17.529 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.297 / 101.649 =	0.003	Low Shear
$M_{c.z.Rd} = f_y.W_{pl.z}/\gamma_{M0}$	235 x 56.13/1	13.191 kN.m	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	18.73 x 235/1 =	440.155 kN	
$n = N_{Ed}/N_{pl.Rd}$	73.909 / 440.155 =	0.168	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	74.59, 11.238, 0.168	74.59 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	74.59 x 235/1	17.529 kN.m	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz},)$	56.13, 7.492, 0.168	56.13 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	56.13 x 235/1	13.191 kN.m	
($M_{y.Ed}$/M_(N.y.Rd+($M_{z.Ed}/M_{N.z.Rd}$)	$(2.942/17.529)^{1.715}+(0.504/13.191)^{1.715}=$	0.051	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x6 =	6	
λy = √A.fy/Ncr	√18.73x235/210.36	1.445	
$N_{b.y.Rd} = Area.c.f_y/\gamma_{M1}$	18.73x0.396x235/10/1 =	174.482 kN	Curve a
Lez = Kz.Lz	1x6 =	6	
λz = √A.fy/Ncrz	√18.73x235/111.09	1.99	
$N_{b.z.Rd} = Area.c.f_y/\gamma_{M1}$	18.73x0.225x235/10/1 =	98.992 kN	Curve a

Equivalent Uniform Moment Factors C1, C.mLT, C.mz, and C.my

C_1= fn(M_1, M_2, M_o, ~y,~m)	2.9, 0.5, -3.6, 0.167, -1.238	2.393	Point
C_{mLT}=Max(-0.8α_s, 0.4)	M_h = 2.94, M_s = -1.93, ~y = 0.167, α_s = -0.655	0.524	Table B.3
C_{mz}=Max(0.2(-~y) -0.8α_s, 0.4)	M_h = -0.5, M_s = 0.39, ~y = -0.549, α_s = -0.767	0.724	Table B.3
C_{my}=Max(-0.8α_s, 0.4)	M_h = 2.94, M_s = -1.92, ~y = 0.167, α_s = -0.654	0.523	Table B.3

Lateral Buckling Check M.b.Rd

$M_{b.Rd} = M_{c.y.Rd}$	Section not susceptible to lateral torsional buckling	17.529 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	73.909 / 174.482	0.424	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	73.909 / 98.992	0.747	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	2.942 / 17.529	0.168	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.504 / 13.191	0.038	OK
$k_y y=C_{my}\{1+0.8U_{N.y}\}$		0.700	
$k_z z=C_{mz}\{1+0.8U_{N.z}\}$		1.156	
$k_y z$=0.6 $k_z z$		0.694	
$k_z y$=0.6 $k_y y$		0.420	
$U_{Ny}+k_y y.U_{M.y}+k_y z.U_{M.z}$	0.424+0.700x0.168+0.694x0.038	0.568	OK
$U_{Nz}+k_z y.U_{M.y}+k_z z.U_{M.z}$	0.747+0.420x0.168+1.156x0.038	0.861	OK

Deflection Check - Load Case 4

Deflection Limits ()	Cantilever δ ≤ 6000/120 = 50 mm Live (Case 1)	1.73 mm	OK
	Cantilever δ ≤ 6000/120 = 50 mm D+L+W (Case 4)	15.79 mm	OK