CONQUIP ENGINEERING GROUP

Temporary Works Design

Project Number:	**ZP21-0103**
Project:	**247 Tottenham Court Rd**
Client:	**Deconstruct**
Calculations of Justification:	**Propping Design**
Date:	**11/07/22**

Document control

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Document Checking:

Revision	Date	Notes	Prepared by	Checked by	Approved by
C01	11/07/22	Design Note	GZ	DCG	DCG

AKT II COMMENTS

Document by: **CONQUIP**
Document title: **Design Note - 247 Tottenham Court R**
Document dated: **11/07/22**

akt II

COMMENTS ISSUED: 03/02/2023 COMMENTS BY: WP CHECKED BY:

Contents

Figures

Tables

1 Introduction

Conquip have been asked by Deconstruct to design the propping system for the excavation at 247 Tottenham Court Rd as shown below.

- Level 1 props to be bolted directly onto concrete corbel above the capping beam
- Level 2 props will be fixed onto waling beam.

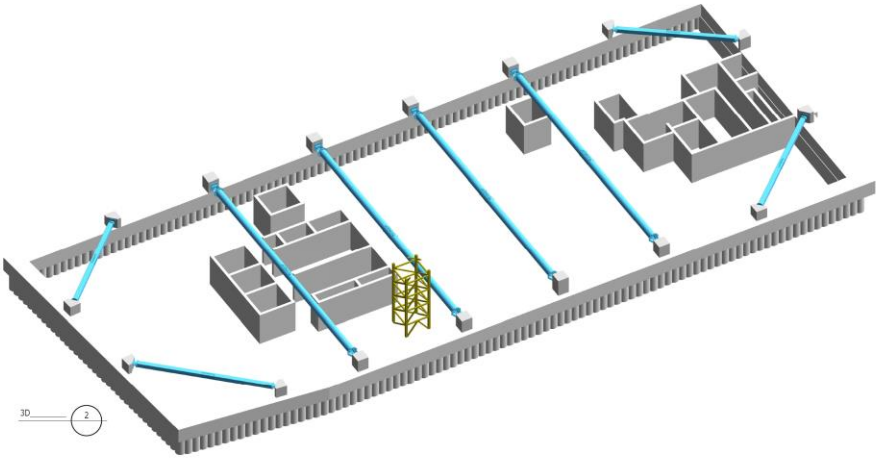

Figure 1: Propping Layout - Level 1

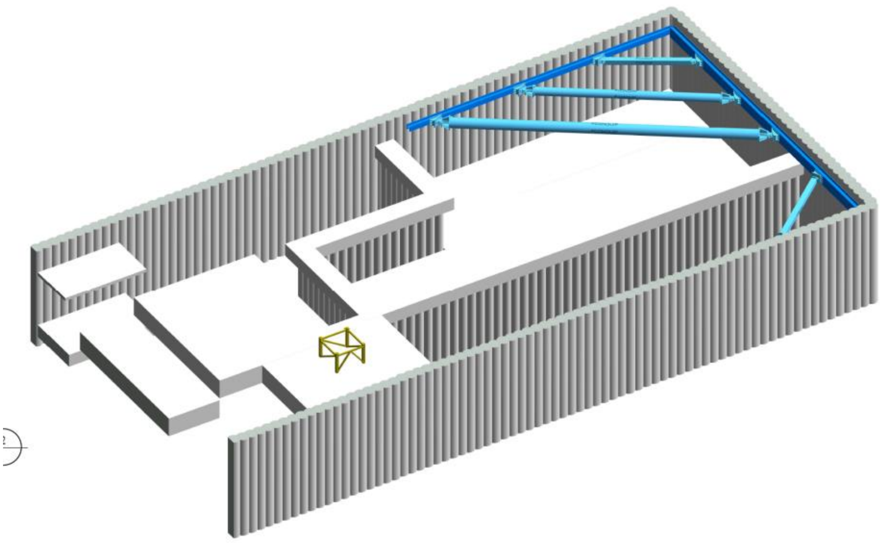

Figure 2 Propping Layout - Level 2

The retaining wall comprises of a Secant Piled Wall and capping beam designed by others. This design covers the following elements:

- Analysis of prop arrangement and prop specification
- Prop Connections

AKT: Temporary works design received covers only these two aspects of the basement propping works. Temporary works review of the capping beam and temporary blisters/thrust blocks are not included in this pack. Contractor to confirm what design review has been completed confirming any additional reinforcement to capping beam over and above the permanent works design.

2 Standard

The structural design has been carried out in conformity with Eurocodes. The props will be designed in ultimate limit state (ULS) and the limit of the deflection to the criteria L/250 at serviceability limit state (SLS).

Standards

- BS EN 1991- Eurocode Actions on Structures
- BS EN 1992-1-1 2008 - Rules for buildings
- BS EN 1993-1-1 Design of Steel Structures to Eurocode
- BS EN 1993-1-8 2005 - Design of joints
- BS EN 1992-4:2018 – Design of fastenings for use in concrete

Guides

- Joints in steel construction: Moment-resisting joints to Eurocode 3
- C760 - Guidance on embedded retaining wall design

3 Information Received

This propping scheme has been designed based on the following documentation / information provided:

- 2176-A2S-XX-XX-CA-Y-0001-00 TCR Pile Wall Design
- 4190-AKT-ZZ-ZZ-M3-S-00001_sam
- 4190-AKT-XX-XX-DR-S-00012
- 4190-AKT-ZZ-00-DR-S-21000
- 4190-AKT-ZZ-00-DR-S-21001
- 4190-AKT-ZZ-B1-DR-S-20990
- 4190-AKT-ZZ-B2-DR-S-20950
- 4190-AKT-ZZ-B2-DR-S-20980
- 4190-AKT-ZZ-ZZ-DR-S-25100
- 4190-AKT-ZZ-ZZ-DR-S-25101
- 4190-AKT-ZZ-ZZ-DR-S-25200

4 Loading

4.1 Load Cases

CONQUIP have taken guidance from CIRIA C760 and have considered the following load cases for the temporary propping design:

- Load Case 1 – $\gamma_g G_k + \gamma_g G_{k,GEO} + \gamma_Q \Psi_0 Q_{k,temp}$
- Load Case 2 – $\gamma_g G_k \xi + \gamma_g G_{k,GEO} \xi + \gamma_Q Q_{k,temp}$
- Load Case 3 – $G_k + G_{k,GEO} + \Psi_0 Q_{k,temp} + Q_{k,accidental}$

From EC0, the following partial factors have been adopted:

- $\gamma_g = 1.35$
- $\gamma_Q = 1.50$
- $\Psi_0 = 0.60$
- $\xi = 0.925$

4.2 Design loadings

Deconstruct have provided design loading applied to the capping beam and waling beam.

Design Section	SLS Temporary Prop Forces (kN/m)		SLS permanent Slab Forces (kN/m)		
	Temp Prop 1	Temp Prop 2	B2 Level	B1 Level	GF Level
SW01-A	108	-	-	123	110
SW02-A	132	-	112	186	128
SW03-A	119	148	271	403	100
SW04-A	119	148	271	403	100
SW01-B	117	-	-	138	113
SW03-B	142	225	519	437	123
SW05-B	142	225	519	437	123
Contiguous (B1-B2)	55	-	-	91	-

AKT: Better quality printout to be provided or clearer labels to capping beam loading references. Alternatively attach and reference the relevant drawing applicable where loading information has been extracted

FIGURE 3 DESIGN LOADINGS

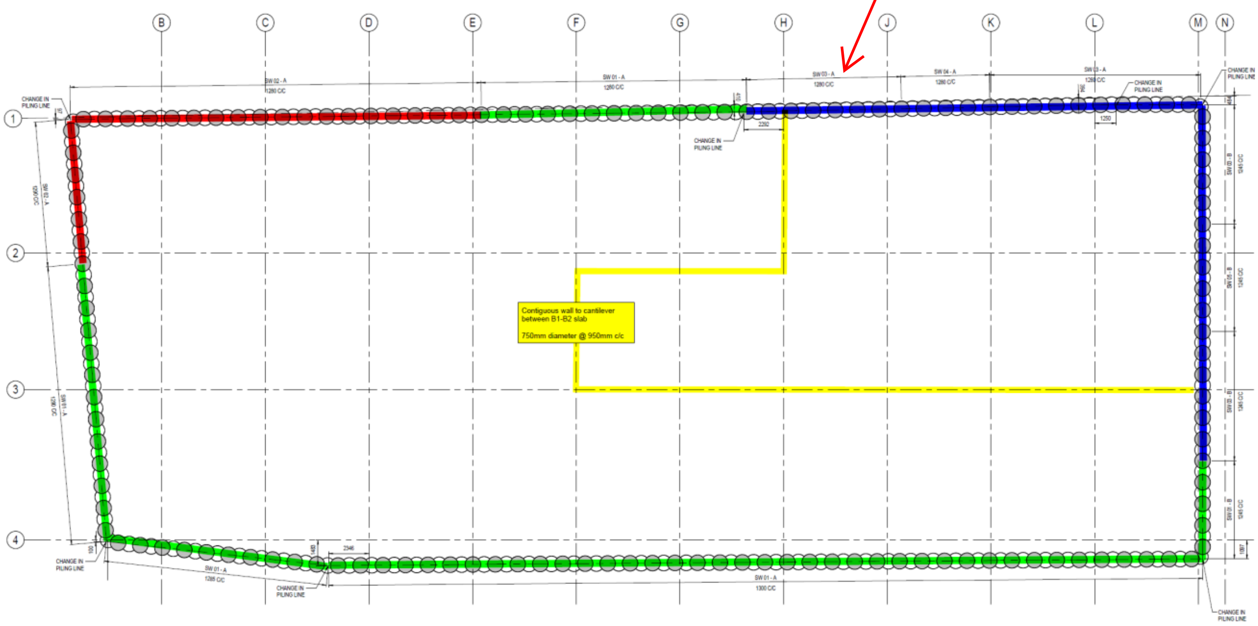

FIGURE 4 DESIGN SECTIONS

4.3 Thermal Loads

Thermal loading will be considered for the design of the props in accordance with CIRIA C760, whereby:

- $Q_{k,temp} = \alpha \Delta t E A \beta$
- $\alpha = 12x10^{-6}$ /°C
- $\Delta t = 15°C$ (assumed)
- E = 210kN/mm²
- A = 24301mm² (400s), 29858 (600s), 37498mm² (800s)
- β = 0.50 (Stiff wall in stiff ground)

Based on the above thermal loads will be considered:

- Conquip 400S – 459.29 kN
- Conquip 600S – 564.32 kN
- Conquip 800s – 708.71 kN

AKT: Is there a relevant codified clause where this accidental load has been taken from?

4.4 Accidental Loads

An accidental load of 10kN at prop midspan is considered on all the props.

5 Construction Sequence

AKT: Are there accompanying sketches outlining the stages of construction showing what is installed and when.

Are raking props installed in advance of piling and install of cross site temporary props.

- Install secant piles from a platform
- Construct Capping beam and corbel
- Install temporary props level 1 at Corbel level
- Excavate to formal level B1
- Install secant piles and Construct Capping beam
- Install temporary props level 2 to waling beam
- Excavate to formal level B2
- Cast basement slab B2
- Cast liner wall and permanent structure from basement B2
- Cast basement slab B1
- Remove temporary props level 2
- Cast liner wall and permanent structure from basement B1
- Cast ground floor
- Remove temporary props level 1

Details to be confirmed by the contractor.

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www.cqegroup.com

6 Props Design Summary

6.1 Frame analysis

A model in Tekla Structural Designer is used to design props. The design elements are shown in the following figure:

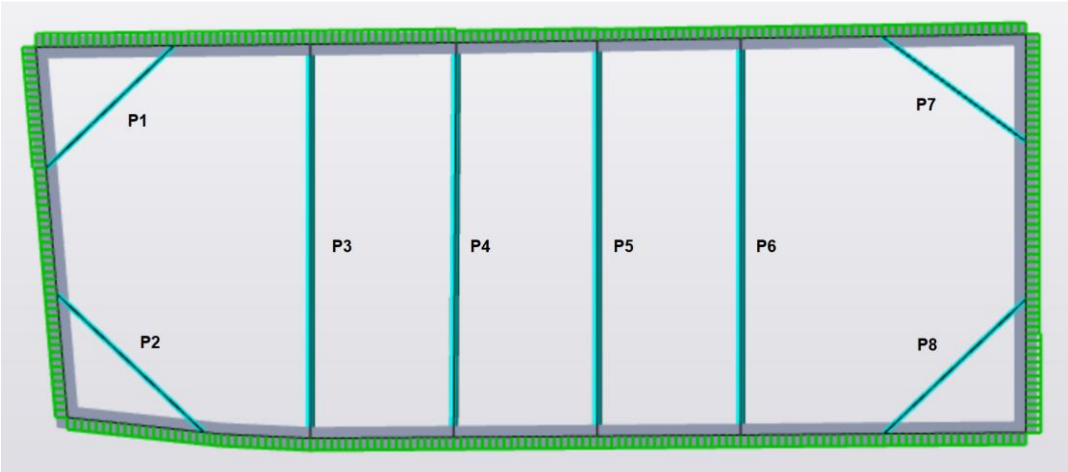


FIGURE 5 LEVEL 1 - DESIGN COMPONENT LABELS

The following cross-sections are used:

- Props 1,2,7 & 8 - SHS 400x16 in S355
- Props 3,4,5 & 6 - CHS 610x16 in S355

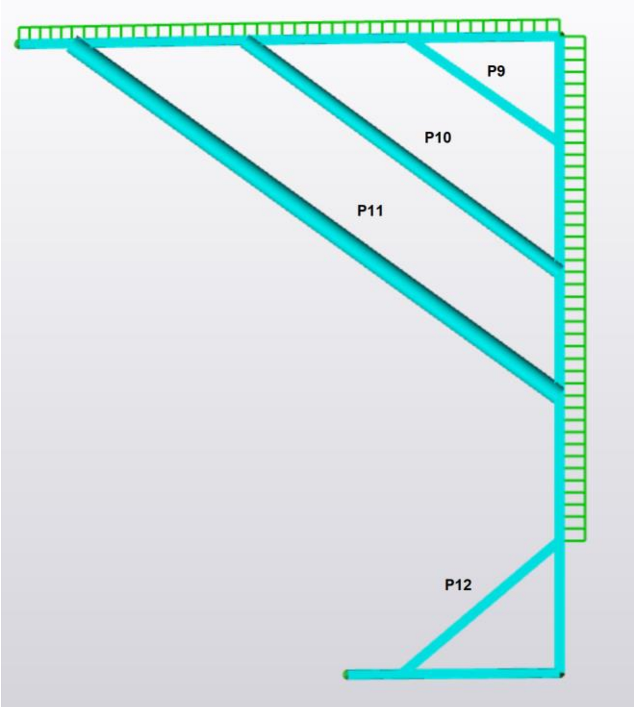


FIGURE 6 DESIGN COMPONENT LABELS - LEVEL 2

- Prop 9 & 12 - SHS 400x16 in S355
- Prop 10 - CHS 610x16 in S355
- Props 11 - CHS 762x16 in S355

6.2 Loads and combinations

6.2.1 Load combinations, prop loads and specification summary

FIGURE 7 LEVEL 1 - PROP AXIAL LOAD - SLS

TABLE 1 LEVEL 1 - LOAD COMBINATION

Prop number	Strut Length (m)	Prop type	Axial Loads (kN)					
			Thermal loads (kN)	Prop axial (kN)	LC1	LC2	LC3	SLS comb.
P1	10.623	400s	459.29	1475.9	2405.83	2533.81	1751.47	1935.19
P2	12.092	400s	459.29	1021.6	1792.52	1965.94	1297.17	1480.89
P3	24.222	600s	564.32	1236	2176.49	2391.48	1574.59	1800.32
P4	24.254	600s	564.32	1081.4	1967.78	2198.23	1419.99	1645.72
P5	24.287	600s	564.32	985.9	1838.85	2078.86	1324.49	1550.22
P6	24.413	600s	564.32	1032.6	1901.90	2137.23	1371.19	1596.92
P7	10.433	400s	459.29	1428.6	2341.97	2474.69	1704.17	1887.89
P8	11.853	400s	459.29	1485.7	2419.06	2546.06	1761.27	1944.99

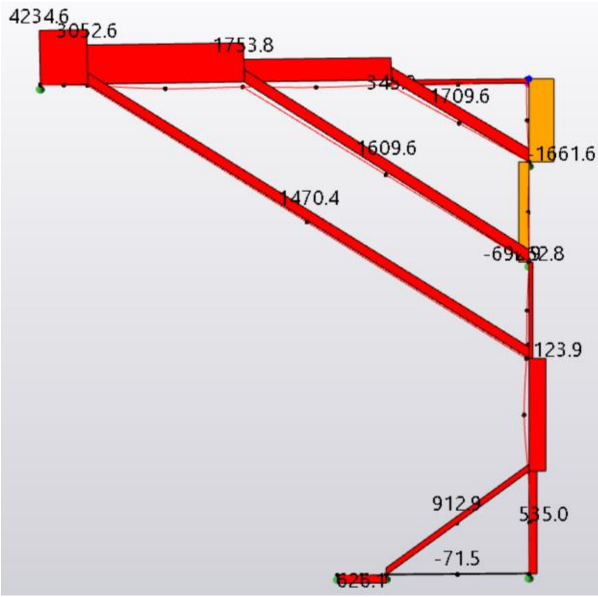


FIGURE 8 LEVEL 2 - PROP AXIAL LOAD – SLS

TABLE 2 LEVEL 2 - LOAD COMBINATION

			Axial Loads (kN)					
Prop number	Strut Length (m)	Prop type	Thermal loads (kN)	Prop axial (kN)	LC1	LC2	LC3	SLS comb.
P9	7.556	400s	459.29	1709.6	2721.32	2825.94	1985.17	2168.89
P10	15.628	600s	564.32	1609.6	2680.85	2858.48	1948.19	2173.92
P11	23.983	800s	708.71	1470.4	2622.88	2901.07	1895.63	2179.11
P12	8.482	400s	459.29	912.9	1645.78	1830.06	1188.47	1372.19

6.3 Prop checks

The prop loading calculations for each type of prop are shown below:

Prop 3 - 600S - CHS 610x16 – Appendix 01
Prop 8 - 400S - SHS 400x16 – Appendix 02
Prop 9 - 400S - SHS 400x16 – Appendix 03
Prop 10 - 600S - SHS 400x16 – Appendix 04
Prop 11 - 800S - CHS 762x16 – Appendix 05

6.4 Prop Pre-Loads

The pre-load for the props is taken as the higher of 15% of the SLS Prop force or the equivalent force of 7.5^{0}C thermal expansion. This is shown in 'kN' and 'BAR'.

TABLE 3 PRE-LOAD OF PROPS

Prop No.	SLS Force (kN)	Pre-Load (kN)	Pre-Load (BAR)
P1	1934.69	290.20	81.26
P2	1486.49	229.65	64.30
P3	1786.92	282.16	79.00
P4	1641.32	282.16	79.00
P5	1566.92	282.16	79.00
P6	1596.92	282.16	79.00
P7	1889.59	283.44	79.36
P8	1943.49	291.52	81.63
P9	2168.89	325.33	91.09
P10	2173.92	326.09	91.31
P11	2179.11	326.87	91.52
P12	1372.19	229.65	64.30

6.5 Waling Beam checks

The waling beam shall be fixed to the piles to provide lateral support for the propping system and any gaps shall be packed to evenly distribute the loadings. A check on the capacity of the waling beam is provided in the appendix.

7 Deflection & Stiffness Checks

7.1 Deflection Check

The following figures show the maximum anticipated deflection under the SLS loading conditions. The maximum deflection is calculated as 7.0mm and 15.9mm of the capping beam and waling beam respectively. The structural engineer must determine if this is satisfactory.

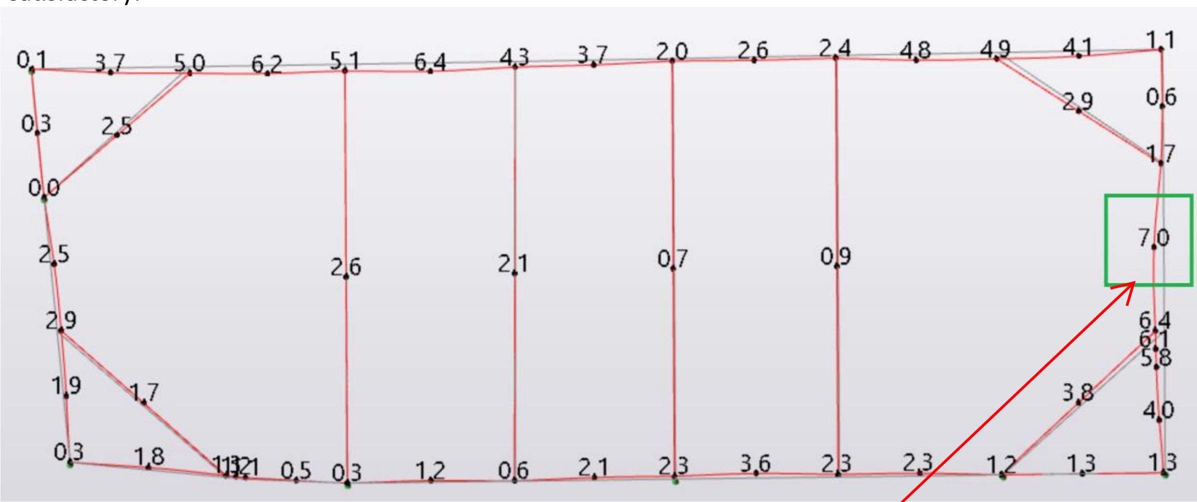


FIGURE 9 DEFLECTION OF CAPPING BEAM

WP: Is this a concern as movements in the temporary condition at capping beam level locally here are at 7mm which is RED trigger level.

WP: Is this a concern as movements in the temporary condition locally here are reaching levels of RED trigger leve (16mm).

Reviewing the monitoring action plan drawing assume this includes permanent works movements. There may not be sufficient redundancy for both temporary and permanent works movements combined?

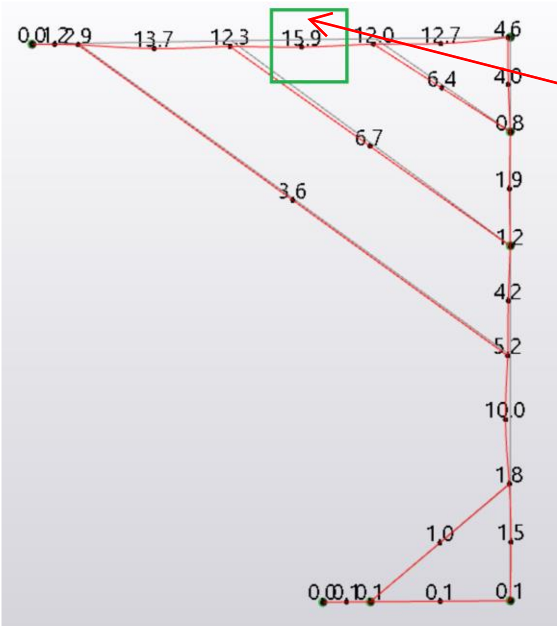


Figure 10 Deflection of waling beam

7.2 Stiffness Check

7.2.1 Level 1

The average stiffness along each gridline is calculated in the following tables. The stiffness of each prop is calculated as:

$$k = \frac{2EA}{L} sin^2(\theta)$$

$E = 2.1\ x\ 10^8\ kN/m^2$

Table 4 Stiffness of wall along gridlines 1 & 9

Prop No.	Prop Type	Area (m^2)	Strut Length (m)	Strut inclination to horizontal ϕ (°)	Stiffness (kN/m)
P1	400s	0.0243	10.623	51	580436
P2	400s	0.0243	12.092	42	377980
				Total Stiffness (kN/m)	958416
				Wall Length (m)	17.0
				Average Stiffness (kN/m/m)	56377

Table 5 Stiffness of wall along gridlines A & N

Prop No.	Prop Type	Area (m^2)	Strut Length (m)	Strut inclination to horizontal ϕ (°)	Stiffness (kN/m)
P1	400s	0.0243	10.623	43	446882
P3	600s	0.0299	24.222	90	517726
P4	600s	0.0299	24.254	90	517043
P5	600s	0.0299	24.287	90	516340
P6	600s	0.0299	24.413	90	513676
P7	400s	0.0243	10.433	37	354316
				Total Stiffness (kN/m)	2,865,983
				Wall Length (m)	57.0
				Average Stiffness (kN/m/m)	50280

7.2.2 Level 2

Table 6 Stiffness of wall along gridlines 1 & 9

Prop No.	Prop Type	Area (m^2)	ut Length (	Strut inclinatio	Stiffness (kN/m)
P9	400s	0.0243	7.556	56	928351
P10	600s	0.0299	15.628	58	577095
P11	800s	0.0375	23.983	57	461888
P12	400s	0.0299	8.482	43	687667
				Total Stiffness (kN/m)	2655001
				Wall Length (m)	20.5
				Average Stiffness (kN/m/m)	129512

Table 7 Stiffness of wall along gridlines A & N

Prop No.	Prop Type	Area (m^2)	ut Length (	Strut inclinatio	Stiffness (kN/m)
P9	400s	0.0243	7.556	35	444372
P10	600s	0.0299	15.628	33	238026
P11	800s	0.0375	23.983	34	205342
				Total Stiffness (kN/m)	887739
				Wall Length (m)	20.0
				Average Stiffness (kN/m/m)	44387

8 Bolted Connection and shear stop Design

AKT: Have Deconstruct reviewed prop forces provided to determined the corbel/thrust block design above the capping beam and also the subsequent temporary works design of the capping beam.

8.1 Determine Shear Loading

8.1.1 Level 1 Props

AKT: For diagrammatic purposes the design principles of the plate should include a sketch for clarity.

The strut base plates are to be fixed at 90^0 to the corbels through bolted connections. Bolts are to take self-weight of the props and accidental load only.

Table 8 Shear component due to self-weight

Prop No.	Base Plate Type	Prop Type	self-weight (kN)	Accidental load (kN)	Shear component at each end
P5	1	600s	74.59	10	47.29

Self-weight = 325kg/m = 3.19kN/m
Length of prop= 24.287m
24.287m*3.19kN/m= 77.48kN
Accidental load= 10kN
Shear component at each end = 77.48kN/2 + 10kN = **48.74kN**
4No. M24 bolts per connection are required for all Level 1 props. **Minimum embedment depth= 210mm**. See relevant Appendix.

8.1.2 Level 2 Props

The shear forces to be resisted by the bolts are displayed in the table below. The End 1 side relates to connections along gridlines A & N. The End 2 relates to the other end of the strut along gridlines 1 and 9. The coefficient of friction is taken as $\mu = 0.1$

TABLE 9 END 1 - SHEAR COMPONENT DUE TO PROP FORCE

Prop number	Base Plate Type	ULS Strut Force, F (kN)	Strut Angle to horizontal	Shear component from strut, Fx (kN)	Normal component from strut, Fy (kN)	Friction Resistance, µ x Fy (kN)	Net Shear Component, Fx,nett (kN)	Type of welded shear stop
P9	2	2825.94	35	2314.87	1620.89	162.09	2152.79	3
P10	2	2858.48	33	2397.32	1556.84	155.68	2241.64	3
P11	2	2901.07	34	2405.10	1622.26	162.23	2242.87	3
P12	2	1830.06	47	1248.10	1338.42	133.84	1114.26	2

TABLE 10 SHEAR COMPONENT DUE TO PROP FORCE

Prop number	Base Plate Type	ULS Strut Force, F (kN)	Strut Angle to	Shear component from strut, Fx (kN)	Normal component from strut, Fy (kN)	Friction Resistance,	Net Shear Component,	Type of welded shear stop
P9	2	2825.94	56	1580.25	2342.81	234.28	1345.96	2
P10	2	2858.48	58	1514.76	2424.13	242.41	1272.35	2
P11	2	2901.07	57	1580.04	2433.04	243.30	1336.73	2
P12	2	1830.06	43	1338.42	1248.10	124.81	1213.61	2

FIGURE 11 CAPACITY OF SHEAR STOPS

Shear Stop Name	ULS Capacity (kN)
Type 2	2000
Type 3	3000

AKT: How do shear keys connect/transfer loads into the secant wall? Please provide simplified diagrams to illustrate how this is intended to work.

9 Welded Shear Key Checks

Welded shear keys are required to prevent the waling beam from sliding.

9.1 Calculate Shear Resistance

The shear keys are 356 x 368 x 202 UC – 350mm Long. The design resistance is the lower of the Concrete strength, the shear strength of the shear key, and the welded connection to the waling beam. The concrete strength of the piles and grouting required is assumed to be at least C28/35.

9.1.1 Concrete Strength (EC2 6.5.2 (1))

Welded shear key	356 x 368 x 202 UC – 350mm long		
Concrete strength - C28/35	28		
deff, embedded length of shear key in concrete	250	mm	
bs, height of shear key flange	374.7	mm	
fcd= (0.85*fck/1.5)	15.87	Mpa	
VRd= Deff*bs*fcd	1486.31	kN	

9.1.2 Shear Key Resistance (EC3 6.2.6 (1))

Shear key resistance		
Av, shear area	6753	mm
fy (S355)	355	
γM0	1	
VRd= Av*(355/3^0.5)γM0/1000	1384.09	

9.1.3 Welded shear key connection (EC3.1.8 4.5.3)

L, length of weld connection (500mm) *2 (two sides)	1000	mm
Fw,l,Rd, strength of 8mm fillet weld	1.35	kN/mm
VRd	1350	kN

Therefore, ULS capacity per shear key = 1350kN

9.2 Calculate Shear Keys required

The shear keys required to resist sliding in each direction are shown below:

TABLE 11 FORCES TO BE RESISTED BY SHEAR KEYS ALONG GRIDLINE A & N – NORTH SIDE

Prop No.	ULS Strut Force, F (kN)	Strut Angle to horizontal	Shear component from strut, Fx (kN)	Normal component from strut, Fy (kN)	Friction Resistance, µ x Fy (kN)	Net Shear Component, Fx,nett (kN)
P9	2825.94	35	2314.87	1620.89	162.09	2152.79
P10	2858.48	33	2397.32	1556.84	155.68	2241.64
P11	2901.07	34	2405.10	1622.26	162.23	2242.87
					Shear from waler (kN)	487.08
					Total shear force (kN)	*7124.37*
					Resistance per Shear key (kN)	1350
					Shear keys Required	6

FIGURE 12 FORCES TO BE RESISTED BY SHEAR KEYS ALONG GRIDLINE A & N – SOUTH SIDE

Prop No.	ULS Strut Force, F (kN)	Strut Angle to horizontal	Shear component from strut, Fx (kN)	Normal component from strut, Fy (kN) Component,	Friction Resistance, µ x Fy (kN)	Net Shear Component, Fx,nett (kN)
P12	1830.06	47	1248.10	1338.42	133.84	1114.26
					Shear from waler (kN)	108
					Total shear force (kN)	*1222.26*
					Resistance per Shear key (kN)	1350
					Shear keys Required	2

10 Residual Risk Assessment

The contractor should note the below risks relating to this scheme. A full risk assessment should be carried out by the contractor to determine further risks specific to the particular site.

- Overdig – exceeding dig levels stated in the pile wall design could lead to excessive loading of the piles/props that could lead to excessive deflection or even failure.
- Prop installation/removal – Props should be installed and removed at suitable stages as determined by the pile wall design – working at height should be avoided if possible. Prop installation/removal should be carried out by competent individuals using suitable lifting equipment and following installation guides provided by Conquip. The installation should be checked and signed off by a qualified engineer.
- Striking Props during excavation – Care should be taken to avoid striking props during the excavation of the basement and installation of the permanent structure. Any striking of props should be reported to Conquip to be assessed. An accidental loading of 10kN at prop centre is applied for design.
- Failure by sliding of waler beam: Waler beam to be welded to the shear keys that will be fixed into the pile wall.
- On site welding – for fixing the shear keys to the waling beam, on site welding is required. Contractor to ensure on site welding is carried out by a qualified welder.
- Packing behind waling beam – Contractor to ensure suitable concrete is used to pack any gaps behind the waling beam to ensure evenly load distribution from the piled wall to the waling beam.
- Grouting of secant pile – Contractor to ensure suitable concrete is used to grout gaps after fixing the shear keys into the secant pile to ensure the shear force can be evenly transferred from the waling beam to the piled wall.

Appendix 01 – Verification of Prop 3 – 600s

Self-weight of prop= 3.19kN/m

$$Bending\ moment\ due\ to\ self\ weight = \frac{wl^2}{8} = \frac{1.35 * 3.19 * 24.222^2}{8} = 315.83kNm$$

$$Bending\ moment\ due\ to\ accidental\ load = \frac{wl}{4} = \frac{10 * 24.222}{4} = 60.56kNm$$

Add second order effect of moment due to deflection:
Deflection at midspan: $\delta = \frac{5wl^4}{384EI} = \frac{5\times3.19\times24.222^4}{384\times2.77\times10^5} = 0.0516m$
Bending moment due to deflection: $2391.48 \times 0.0516 = 123.4\ kNm$

Total BM at mid-span = $315.83 + 60.56 + 123.4 = 499.79\ kNm$

$$Maximum\ shear\ force = \frac{wl}{2} + 10kN = \frac{1.35*3.19*24.222}{2} + 10kN = 60.16kN$$

STEEL MEMBER DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.4.08

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

Design section 1

Section details

Section type Euro CHS 610.0x16.0 (BS EN 10210)

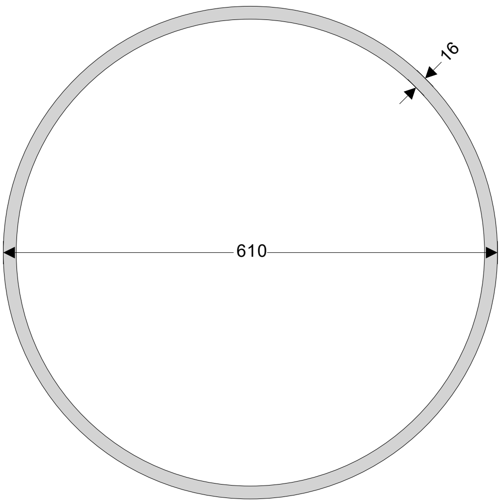
Steel grade - EN 10210-1:2006 S355H

Nominal thickness of element t_{nom} = t = **16** mm

Nominal yield strength f_y = **355** N/mm^2

Nominal ultimate tensile strength f_u = **470** N/mm^2

Modulus of elasticity E = **210000** N/mm^2



Euro CHS 610.0x16.0 (BS EN 10210)
Diameter, d, 610 mm
Mass of section, Mass, 234.4 kg/m
Section thickness, t, 16 mm
Area of section, A, 29858 mm^2
Radius of gyration about y-axis, i_y, 210.087 mm
Radius of gyration about z-axis, i_z, 210.087 mm
Elastic section modulus about y-axis, $W_{el.y}$, 4320702 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 4320702 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 5646741 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 5646741 mm^3
Second moment of area about y-axis, I_y, 1317814225 mm^4
Second moment of area about z-axis, I_z, 1317814225 mm^4

Analysis results

Design bending moment - Major axis $M_{y,Ed}$ = **499.79** kNm

Design shear force - Major axis $V_{y,Ed}$ = **60.21** kN

Design axial compression force N_{Ed} = **2391.48** kN

Restraint spacing

Major axis lateral restraint L_y = **24222** mm

Minor axis lateral restraint L_z = **24222** mm

Torsional restraint L_T = **0** mm

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

Tubular sections - Table 5.2 (sheet 3 of 3)

$d / t = 38.1 = 57.6 \times \varepsilon^2 <= 70 \times \varepsilon^2$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force N_{Ed} = **2391.5** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **10599.5** kN

$N_{Ed} / N_{c,Rd}$ = **0.226**

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,y} = L_{y\_s1}$ = **24222** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **4655.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **1.509**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 α_y = **0.21**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **1.776**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.369**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **3908** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.612**

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,z} = L_{z\_s1}$ = **24222** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **4655.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **1.509**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 α_z = **0.21**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **1.776**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.369**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **3908** kN

$N_{Ed} / N_{b,z,Rd}$ = **0.612**

PASS - Design buckling resistance exceeds design compression

Check shear - Section 6.2.6

Design shear force $V_{y,Ed}$ = **60.2** kN

Shear area - cl 6.2.6(3) $A_v = 2 \times A / \pi$ = **19008** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **3895.9** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.015**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **499.8** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **2004.6** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.249**

PASS - Design bending resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Normal force to plastic resistance force ratio $n = N_{Ed} / N_{pl,Rd}$ = **0.226**

Reduced plastic moment resistance - Eq.6.39 $M_{N,y,Rd} = M_{pl,y,Rd} \times (1 - n^{1.7})$ = **1845.1** kNm

$M_{y,Ed} / M_{N,y,Rd}$ = **0.271**

PASS - Reduced bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 C_{my} = **1.000**

C_{mz} = **1.000**

C_{mLT} = **1.000**

Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **2004.6** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **2004.6** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **10599.5** kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **1.490**

$k_{zy} = 0.6 \times k_{yy}$ = **0.894**

χ_{LT} = **1.000**

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.983**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.835**

PASS - Combined bending and compression checks are satisfied

Appendix 02 – Verification of Prop 8 - 400s

Self-weight of prop= 2.45kN/m

$$Bending\ moment\ due\ to\ self\ weight = \frac{wl^2}{8} = \frac{1.35 * 2.45 * 11.853^2}{8} = 58.09kNm$$

$$Bending\ moment\ due\ to\ accidental\ load = \frac{wl}{4} = \frac{10 * 11.853}{4} = 29.63kNm$$

Add second order effect of moment due to deflection:
Deflection at midspan: $\delta = \frac{5wl^4}{384EI} = \frac{5\times2.45\times11.853^4}{384\times2.77\times10^5} = 0.0023m$
Bending moment due to deflection: $2546.06 \times 0.0023 = 5.86\ kNm$

Total BM at mid-span = 58.09 + 29.63 + 5.86 = 93.58kNm

$$Maximum\ shear\ force = \frac{wl}{2} + 10kN = \frac{1.35*2.45*11.853}{2} + 10kN = 30.0kN$$

STEEL MEMBER DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.4.08

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

Design section 1

Section details

Section type SHS 400x400x16.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))

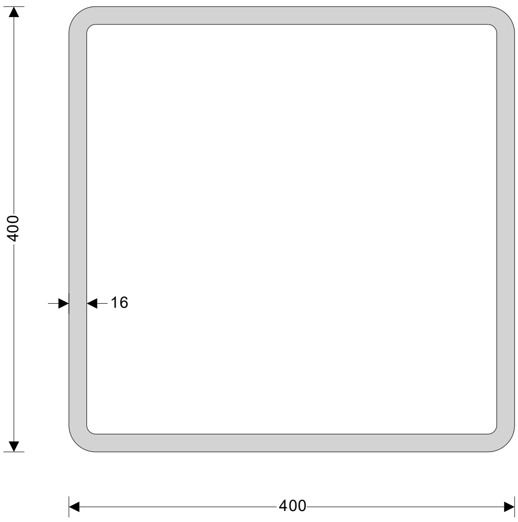
Steel grade - EN 10210-1:2006 S355H

Nominal thickness of element t_{nom} = t = **16** mm

Nominal yield strength f_y = **355** N/mm^2

Nominal ultimate tensile strength f_u = **470** N/mm^2

Modulus of elasticity E = **210000** N/mm^2



SHS 400x400x16.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))
Section depth, h, 400 mm
Section breadth, b, 400 mm
Mass of section, Mass, 190.8 kg/m
Section thickness, t, 16 mm
Area of section, A, 24301 mm^2
Radius of gyration about y-axis, i_y, 156.27 mm
Radius of gyration about z-axis, i_z, 156.27 mm
Elastic section modulus about y-axis, $W_{el.y}$, 2967212 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 2967212 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 3484403 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 3484403 mm^3
Second moment of area about y-axis, I_y, 593442375 mm^4
Second moment of area about z-axis, I_z, 593442375 mm^4

Analysis results

Design bending moment - Major axis $M_{y,Ed}$ = **93.58** kNm

Design shear force - Major axis $V_{y,Ed}$ = **30** kN

Design axial compression force N_{Ed} = **2546.06** kN

Restraint spacing

Major axis lateral restraint L_y = **11853** mm

Minor axis lateral restraint L_z = **11853** mm

Torsional restraint L_T = **0** mm

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)

Width of section c = h - 3 × t = **352** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times 2 \times t \times f_y) - 3 \times t / 2] / c, 1)$ = **0.818**

$c / t = 22 = 27 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Internal compression parts subject to compression - Table 5.2 (sheet 1 of 3)

Width of section $c = b - 3 \times t = \mathbf{352}$ mm

$c / t = 22 = 27 \times \varepsilon <= 33 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force $N_{Ed} = \mathbf{2546.1}$ kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = \mathbf{8627}$ kN

$N_{Ed} / N_{c,Rd} = \mathbf{0.295}$

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,y} = L_{y\_s1} = \mathbf{11853}$ mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = \mathbf{8754.7}$ kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{}(A \times f_y / N_{cr,y}) = \mathbf{0.993}$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y = \mathbf{0.21}$

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) = \mathbf{1.076}$

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{}(\phi_y^2 - \bar{\lambda}_y^2)), 1) = \mathbf{0.671}$

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = \mathbf{5786.3}$ kN

$N_{Ed} / N_{b,y,Rd} = \mathbf{0.44}$

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,z} = L_{z\_s1} = \mathbf{11853}$ mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = \mathbf{8754.7}$ kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{}(A \times f_y / N_{cr,z}) = \mathbf{0.993}$

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_z = \mathbf{0.21}$

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) = \mathbf{1.076}$

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{}(\phi_z^2 - \bar{\lambda}_z^2)), 1) = \mathbf{0.671}$

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \mathbf{5786.3}$ kN

$N_{Ed} / N_{b,z,Rd} = \mathbf{0.44}$

PASS - Design buckling resistance exceeds design compression

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t = \mathbf{368}$ mm $\eta = \mathbf{1.000}$

$h_w / t = 23 = 28.3 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed} = \mathbf{30}$ kN

Shear area - cl 6.2.6(3) $A_v = A \times h / (b + h) = \mathbf{12151}$ mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{}(3)) / \gamma_{M0} = \mathbf{2490.4}$ kN

$V_{y,Ed} / V_{c,y,Rd} = \mathbf{0.012}$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **93.6** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **1237** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.076**

PASS - Design bending resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Normal force to plastic resistance force ratio $n = N_{Ed} / N_{pl,Rd}$ = **0.295**

$a_w = \min((A - 2 \times b \times t) / A, 0.5)$ = **0.473**

Reduced plastic moment resistance - Eq.6.39 $M_{N,y,Rd} = M_{pl,y,Rd} \times \min((1 - n) / (1 - 0.5 \times a_w), 1)$ = **1142.2** kNm

$M_{y,Ed} / M_{N,y,Rd}$ = **0.082**

PASS - Reduced bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 C_{my} = **1.000**

C_{mz} = **1.000**

C_{mLT} = **1.000**

Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **1237** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **1237** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **8627** kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **1.349**

$k_{zy} = 0.6 \times k_{yy}$ = **0.809**

χ_{LT} = **1.000**

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.542**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.501**

PASS - Combined bending and compression checks are satisfied

Appendix 03 - Verification of Prop 9 - 400s

Self-weight of prop= 2.45kN/m

$$Bending\ moment\ due\ to\ self\ weight = \frac{wl^2}{8} = \frac{1.35 * 2.45 * 7.556^2}{8} = 23.6kNm$$

$$Bending\ moment\ due\ to\ accidental\ load = \frac{wl}{4} = \frac{10 * 7.556}{4} = 18.89kNm$$

Add second order effect of moment due to deflection:

Deflection at midspan: $\delta = \frac{5wl^4}{384EI} = \frac{5\times2.45\times7.556^4}{384\times2.77\times10^5} = 0.0004m$

Bending moment due to deflection: $2825.94 \times 0.0004 = 1.13\ kNm$

Total BM at mid-span = 23.6 + 18.89 + 1.13 = 43.62kNm

$$Maximum\ shear\ force = \frac{wl}{2} + 10kN = \frac{1.35*2.45*7.556}{2} + 10kN = 22.50kN$$

STEEL MEMBER DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.4.08

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

Design section 1

Section details

Section type SHS 400x400x16.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))

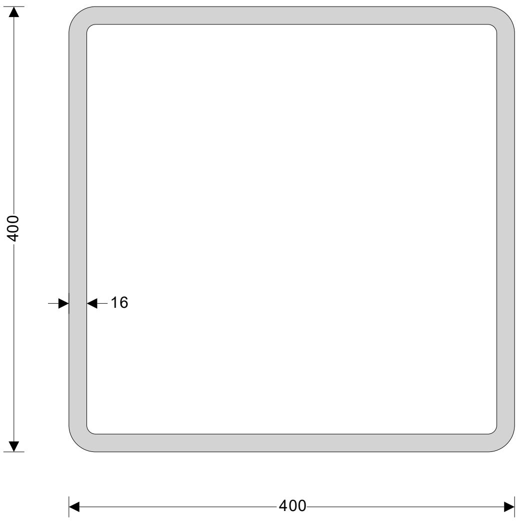
Steel grade - EN 10210-1:2006 S355H

Nominal thickness of element t_{nom} = t = **16** mm

Nominal yield strength f_y = **355** N/mm^2

Nominal ultimate tensile strength f_u = **470** N/mm^2

Modulus of elasticity E = **210000** N/mm^2



SHS 400x400x16.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))
Section depth, h, 400 mm
Section breadth, b, 400 mm
Mass of section, Mass, 190.8 kg/m
Section thickness, t, 16 mm
Area of section, A, 24301 mm^2
Radius of gyration about y-axis, i_y, 156.27 mm
Radius of gyration about z-axis, i_z, 156.27 mm
Elastic section modulus about y-axis, $W_{el.y}$, 2967212 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 2967212 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 3484403 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 3484403 mm^3
Second moment of area about y-axis, I_y, 593442375 mm^4
Second moment of area about z-axis, I_z, 593442375 mm^4

Analysis results

Design bending moment - Major axis $M_{y,Ed}$ = **43.62** kNm

Design shear force - Major axis $V_{y,Ed}$ = **22.5** kN

Design axial compression force N_{Ed} = **2825.94** kN

Restraint spacing

Major axis lateral restraint L_y = **7556** mm

Minor axis lateral restraint L_z = **7556** mm

Torsional restraint L_T = **0** mm

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)

Width of section c = h - 3 × t = **352** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times 2 \times t \times f_y) - 3 \times t / 2] / c, 1)$ = **0.853**

$c / t = 22 = 27 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Internal compression parts subject to compression - Table 5.2 (sheet 1 of 3)

Width of section $c = b - 3 \times t =$ **352** mm

$c / t = 22 = 27 \times \varepsilon <= 33 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force $N_{Ed} =$ **2825.9** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} =$ **8627** kN

$N_{Ed} / N_{c,Rd} =$ **0.328**

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,y} = L_{y\_s1} =$ **7556** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **21543.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{}(A \times f_y / N_{cr,y}) =$ **0.633**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y =$ **0.21**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.746**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{}(\phi_y^2 - \bar{\lambda}_y^2)), 1) =$ **0.877**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **7566.9** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.373**

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,z} = L_{z\_s1} =$ **7556** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **21543.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{}(A \times f_y / N_{cr,z}) =$ **0.633**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_z =$ **0.21**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **0.746**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{}(\phi_z^2 - \bar{\lambda}_z^2)), 1) =$ **0.877**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **7566.9** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.373**

PASS - Design buckling resistance exceeds design compression

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t =$ **368** mm $\eta =$ **1.000**

$h_w / t = 23 = 28.3 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed} =$ **22.5** kN

Shear area - cl 6.2.6(3) $A_v = A \times h / (b + h) =$ **12151** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{}(3)) / \gamma_{M0} =$ **2490.4** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.009**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment: $M_{y,Ed}$ = **43.6** kNm

Design bending resistance moment - eq 6.13: $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **1237** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.035**

PASS - Design bending resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Normal force to plastic resistance force ratio: $n = N_{Ed} / N_{pl,Rd}$ = **0.328**

$a_w = \min((A - 2 \times b \times t) / A, 0.5)$ = **0.473**

Reduced plastic moment resistance - Eq.6.39: $M_{N,y,Rd} = M_{pl,y,Rd} \times \min((1 - n) / (1 - 0.5 \times a_w), 1)$ = **1089.6** kNm

$M_{y,Ed} / M_{N,y,Rd}$ = **0.04**

PASS - Reduced bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3: C_{my} = **1.000**

C_{mz} = **1.000**

C_{mLT} = **1.000**

Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1

Characteristic moment resistance: $M_{y,Rk} = W_{pl.y} \times f_y$ = **1237** kNm

Characteristic moment resistance: $M_{z,Rk} = W_{pl.z} \times f_y$ = **1237** kNm

Characteristic resistance to normal force: $N_{Rk} = A \times f_y$ = **8627** kN

Interaction factors: $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **1.162**

$k_{zy} = 0.6 \times k_{yy}$ = **0.697**

χ_{LT} = **1.000**

Interaction formulae - eq 6.61 & eq 6.62: $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.414**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.398**

PASS - Combined bending and compression checks are satisfied

Appendix 04 - Verification of Prop 10 - 600s

Self-weight of prop= 3.19kN/m

$$Bending\ moment\ due\ to\ self\ weight = \frac{wl^2}{8} = \frac{1.35 * 3.19 * 15.628^2}{8} = 131.47kNm$$

$$Bending\ moment\ due\ to\ accidental\ load = \frac{wl}{4} = \frac{10 * 15.628}{4} = 39.07kNm$$

Add second order effect of moment due to deflection:

Deflection at midspan: $\delta = \frac{5wl^4}{384EI} = \frac{5\times3.19\times15.628^4}{384\times2.77\times10^5} = 0.0089m$

Bending moment due to deflection: $2858.48 \times 0.0089 = 25.44\ kNm$

Total BM at mid-span = 131.47 + 39.07 + 25.44 = 195.98kNm

$$Maximum\ shear\ force = \frac{wl}{2} + 10kN = \frac{1.35*3.19*15.628}{2} + 10kN = 43.65kN$$

STEEL MEMBER DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.4.08

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

Design section 1

Section details

Section type Euro CHS 610.0x16.0 (BS EN 10210)

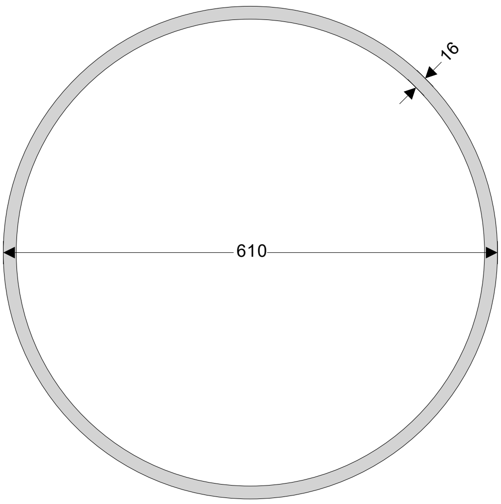
Steel grade - EN 10210-1:2006 S355H

Nominal thickness of element t_{nom} = t = **16** mm

Nominal yield strength f_y = **355** N/mm^2

Nominal ultimate tensile strength f_u = **470** N/mm^2

Modulus of elasticity E = **210000** N/mm^2



Euro CHS 610.0x16.0 (BS EN 10210)
Diameter, d, 610 mm
Mass of section, Mass, 234.4 kg/m
Section thickness, t, 16 mm
Area of section, A, 29858 mm^2
Radius of gyration about y-axis, i_y, 210.087 mm
Radius of gyration about z-axis, i_z, 210.087 mm
Elastic section modulus about y-axis, $W_{el.y}$, 4320702 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 4320702 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 5646741 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 5646741 mm^3
Second moment of area about y-axis, I_y, 1317814225 mm^4
Second moment of area about z-axis, I_z, 1317814225 mm^4

Analysis results

Design bending moment - Major axis $M_{y,Ed}$ = **195.98** kNm

Design shear force - Major axis $V_{y,Ed}$ = **43.65** kN

Design axial compression force N_{Ed} = **2858.48** kN

Restraint spacing

Major axis lateral restraint L_y = **15628** mm

Minor axis lateral restraint L_z = **15628** mm

Torsional restraint L_T = **0** mm

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

Tubular sections - Table 5.2 (sheet 3 of 3)

$d / t = 38.1 = 57.6 \times \varepsilon^2 <= 70 \times \varepsilon^2$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force N_{Ed} = **2858.5** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **10599.5** kN

$N_{Ed} / N_{c,Rd}$ = **0.27**

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,y} = L_{y\_s1}$ = **15628** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **11183.2** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.974**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 α_y = **0.21**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **1.055**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.684**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **7250.4** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.394**

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,z} = L_{z\_s1}$ = **15628** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **11183.2** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **0.974**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 α_z = **0.21**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **1.055**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.684**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **7250.4** kN

$N_{Ed} / N_{b,z,Rd}$ = **0.394**

PASS - Design buckling resistance exceeds design compression

Check shear - Section 6.2.6

Design shear force $V_{y,Ed}$ = **43.7** kN

Shear area - cl 6.2.6(3) $A_v = 2 \times A / \pi$ = **19008** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **3895.9** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.011**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **196** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **2004.6** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.098**

PASS - Design bending resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Normal force to plastic resistance force ratio $n = N_{Ed} / N_{pl,Rd}$ = **0.27**

Reduced plastic moment resistance - Eq.6.39 $M_{N,y,Rd} = M_{pl,y,Rd} \times (1 - n^{1.7})$ = **1788.6** kNm

$M_{y,Ed} / M_{N,y,Rd}$ = **0.11**

PASS - Reduced bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 C_{my} = **1.000**

C_{mz} = **1.000**

C_{mLT} = **1.000**

Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **2004.6** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **2004.6** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **10599.5** kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **1.305**

$k_{zy} = 0.6 \times k_{yy}$ = **0.783**

χ_{LT} = **1.000**

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.522**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.471**

PASS - Combined bending and compression checks are satisfied

Appendix 05 - Verification of Prop 11 - 800s

Self-weight of prop= 3.70kN/m

$$Bending\ moment\ due\ to\ self\ weight = \frac{wl^2}{8} = \frac{1.35 * 3.7 * 23.983^2}{8} = 359.13kNm$$

$$Bending\ moment\ due\ to\ accidental\ load = \frac{wl}{4} = \frac{10 * 23.983}{4} = 59.96kNm$$

Add second order effect of moment due to deflection:
Deflection at midspan: $\delta = \frac{5wl^4}{384EI} = \frac{5\times3.7\times23.983^4}{384\times2.77\times10^5} = 0.058m$

Bending moment due to deflection: $2901.07 \times 0.058 = 168.26\ kNm$

Total BM at mid-span = 359.13 + 59.96 + 168.26 = 195.98kNm

$$Maximum\ shear\ force = \frac{wl}{2} + 10kN = \frac{1.35*3.7*23.983}{2} + 10kN = 69.90kN$$

STEEL MEMBER DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.4.08

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

Design section 1

Section details

Section type Euro CHS 762.0x16.0 (BS EN 10210)

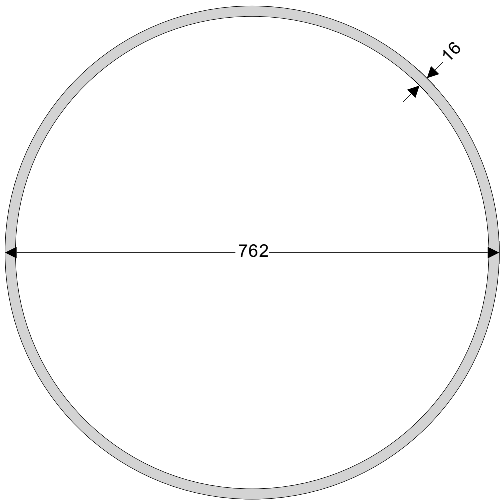
Steel grade - EN 10210-1:2006 S355H

Nominal thickness of element t_{nom} = t = **16** mm

Nominal yield strength f_y = **355** N/mm^2

Nominal ultimate tensile strength f_u = **470** N/mm^2

Modulus of elasticity E = **210000** N/mm^2



Euro CHS 762.0x16.0 (BS EN 10210)
Diameter, d, 762 mm
Mass of section, Mass, 294.4 kg/m
Section thickness, t, 16 mm
Area of section, A, 37498 mm^2
Radius of gyration about y-axis, i_y, 263.811 mm
Radius of gyration about z-axis, i_z, 263.811 mm
Elastic section modulus about y-axis, $W_{el.y}$, 6849693 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 6849693 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 8905621 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 8905621 mm^3
Second moment of area about y-axis, I_y, 2609733031 mm^4
Second moment of area about z-axis, I_z, 2609733031 mm^4

Analysis results

Design bending moment - Major axis $M_{y,Ed}$ = **588.07** kNm

Design shear force - Major axis $V_{y,Ed}$ = **69.9** kN

Design axial compression force N_{Ed} = **2901.07** kN

Restraint spacing

Major axis lateral restraint L_y = **23983** mm

Minor axis lateral restraint L_z = **23983** mm

Torsional restraint L_T = **0** mm

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

Tubular sections - Table 5.2 (sheet 3 of 3)

$d / t = 47.6 = 71.9 \times \varepsilon^2 <= 90 \times \varepsilon^2$ Class 3

Section is class 3

Check compression - Section 6.2.4

Design compression force N_{Ed} = **2901.1** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **13311.8** kN

$N_{Ed} / N_{c,Rd}$ = **0.218**

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,y} = L_{y\_s1}$ = **23983** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **9403.9** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **1.19**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 α_y = **0.21**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **1.312**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.536**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **7141.6** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.406**

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,z} = L_{z\_s1}$ = **23983** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **9403.9** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **1.19**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 α_z = **0.21**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **1.312**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.536**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **7141.6** kN

$N_{Ed} / N_{b,z,Rd}$ = **0.406**

PASS - Design buckling resistance exceeds design compression

Check shear - Section 6.2.6

Design shear force $V_{y,Ed}$ = **69.9** kN

Shear area - cl 6.2.6(3) $A_v = 2 \times A / \pi$ = **23872** mm^2

Design shear resistance - cl 6.2.6(2) $V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **4892.8** kN

$V_{y,Ed} / V_{pl,y,Rd}$ = **0.014**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **588.1** kNm

Design bending resistance moment - eq 6.14 $M_{c,y,Rd} = M_{el,y,Rd} = W_{el.y} \times f_y / \gamma_{M0}$ = **2431.6** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.242**

PASS - Design bending resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Maximum longitudinal stress $\sigma_{y,Ed} = M_{y,Ed} / W_{el.y} + N_{Ed} / A$ = **163** N/mm^2

Limiting longitudinal stress - Eq.6.42 $\sigma_{y,lim} = f_y / \gamma_{M0}$ = **355** N/mm^2

$\sigma_{y,Ed} / \sigma_{y,lim}$ = **0.46**

PASS - Maximum longitudinal stress is less than limiting longitudinal stress

Interaction formula - eq.6.2 $N_{Ed} / N_{c,Rd} + M_{y,Ed} / M_{c,y,Rd} = \mathbf{0.46}$

PASS - Utilisation of combined bending and axial force is acceptable

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 $C_{my} = \mathbf{1.000}$

$C_{mz} = \mathbf{1.000}$

$C_{mLT} = \mathbf{1.000}$

Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1

Characteristic moment resistance $M_{y,Rk} = W_{el.y} \times f_y = \mathbf{2431.6}$ kNm

Characteristic moment resistance $M_{z,Rk} = W_{el.z} \times f_y = \mathbf{2431.6}$ kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y = \mathbf{13311.8}$ kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(0.6 \times \bar{\lambda}_y, 0.6) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = \mathbf{1.244}$

$k_{zy} = 0.8 \times k_{yy} = \mathbf{0.995}$

$\chi_{LT} = \mathbf{1.000}$

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = \mathbf{0.707}$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = \mathbf{0.647}$

PASS - Combined bending and compression checks are satisfied

Appendix 06 - Verification of waling beam

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STEEL MEMBER DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.4.08

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

Design section 1

Section details

Section type UKC 356x406x340 (Tata Steel Advance)

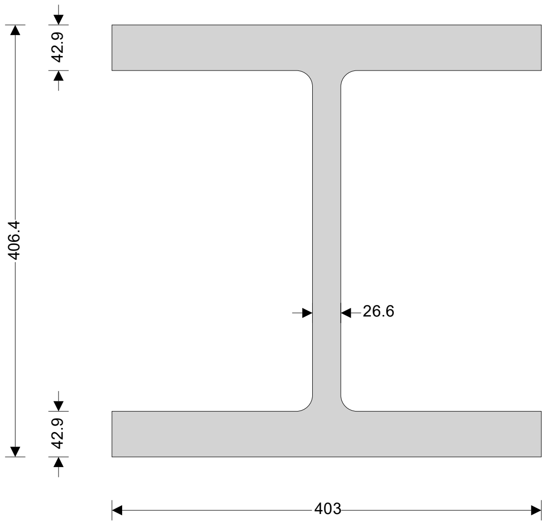
Steel grade - EN 10025-2:2004 S355

Nominal thickness of element t_{nom} = max(t_f, t_w) = **42.9** mm

Nominal yield strength f_y = **335** N/mm^2

Nominal ultimate tensile strength f_u = **470** N/mm^2

Modulus of elasticity E = **210000** N/mm^2



UKC 356x406x340 (Tata Steel Advance)
Section depth, h, 406.4 mm
Section breadth, b, 403 mm
Mass of section, Mass, 339.9 kg/m
Flange thickness, t_f, 42.9 mm
Web thickness, t_w, 26.6 mm
Root radius, r, 15.2 mm
Area of section, A, 43304 mm^2
Radius of gyration about y-axis, i_y, 168.222 mm
Radius of gyration about z-axis, i_z, 104.018 mm
Elastic section modulus about y-axis, $W_{el.y}$, 6030658 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 2325227 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 6999078 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 3543694 mm^3
Second moment of area about y-axis, I_y, 1225429774 mm^4
Second moment of area about z-axis, I_z, 468533140 mm^4

Analysis results

Design bending moment - Major axis $M_{y,Ed}$ = **832.1** kNm

Design shear force - Major axis $V_{y,Ed}$ = **985.3** kN

Design axial compression force N_{Ed} = **4700** kN

Restraint spacing

Major axis lateral restraint L_y = **5700** mm

Minor axis lateral restraint L_z = **5700** mm

Torsional restraint L_T = **0** mm

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.84**

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)

Width of section c = d = **290.2** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **1.000**

$c / t_w = 10.9 = 13 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **173** mm

$c / t_f = 4 = 4.8 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force $N_{Ed} =$ **4700** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} =$ **14506.7** kN

$N_{Ed} / N_{c,Rd} =$ **0.324**

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,y} = L_{y\_s1} =$ **5700** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **78173.2** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{}(A \times f_y / N_{cr,y}) =$ **0.431**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_y =$ **0.34**

Buckling reduction determination factor $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.632**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\phi_y + \sqrt{}(\phi_y^2 - \bar{\lambda}_y^2)), 1) =$ **0.914**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **13254.4** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.355**

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length $L_{cr,z} = L_{z\_s1} =$ **5700** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **29888.9** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{}(A \times f_y / N_{cr,z}) =$ **0.697**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 c

Imperfection factor - Table 6.1 $\alpha_z =$ **0.49**

Buckling reduction determination factor $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **0.864**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\phi_z + \sqrt{}(\phi_z^2 - \bar{\lambda}_z^2)), 1) =$ **0.727**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **10542.7** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.446**

PASS - Design buckling resistance exceeds design compression

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f =$ **320.6** mm $\eta =$ **1.000**

$h_w / t_w = 12.1 = 14.4 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed} =$ **985.3** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **11172** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{}(3)) / \gamma_{M0} =$ **2160.7** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.456**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed}$ = **832.1** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **2344.7** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.355**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - User defined — k_c = **1**

$C_1 = 1 / k_c^2$ = **1**

Poissons ratio — ν = **0.3**

Shear modulus — $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained effective length — $L = 1.0 \times L_{z\_s1}$ = **5700** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **9277.2** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.503**

Limiting slenderness ratio — $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.612**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.959**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **1.000**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.959**

Design buckling resistance moment - eq 6.55 — $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **2248.7** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.37**

PASS - Design buckling resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Bending and axial force check - eq.6.33 & eq.6.34 — $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **1428.4** kN

$N_{Ed} / N_{y,lim}$ = **3.29**

Allowance needs to be made for the effect of the axial force on the plastic resistance moment about the y-y axis

Normal force to plastic resistance force ratio — $n = N_{Ed} / N_{pl,Rd}$ = **0.324**

$a = \min((A - 2 \times b \times t_f) / A, 0.5)$ = **0.202**

Reduced plastic moment resistance - Eq.6.36 — $M_{N,y,Rd} = M_{pl,y,Rd} \times \min((1 - n) / (1 - 0.5 \times a), 1)$ = **1762.6** kNm

$M_{y,Ed} / M_{N,y,Rd}$ = **0.472**

PASS - Reduced bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 — C_{my} = **1.000**

C_{mz} = **1.000**

C_{mLT} = **1.000**

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance — $M_{y,Rk} = W_{pl.y} \times f_y$ = **2344.7** kNm

Characteristic moment resistance — $M_{z,Rk} = W_{pl.z} \times f_y$ = **1187.1** kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y$ = **14506.7** kN

Interaction factors — $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **1.082**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = \mathbf{0.959}$

Interaction formulae - eq 6.61 & eq 6.62

$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = \mathbf{0.755}$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = \mathbf{0.801}$

PASS - Combined bending and compression checks are satisfied

Appendix 07 – Hilti Connection Design

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www.hilti.co.uk

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - 30 Jun 2022	Date:	30/06/2022
Fastening Point:			

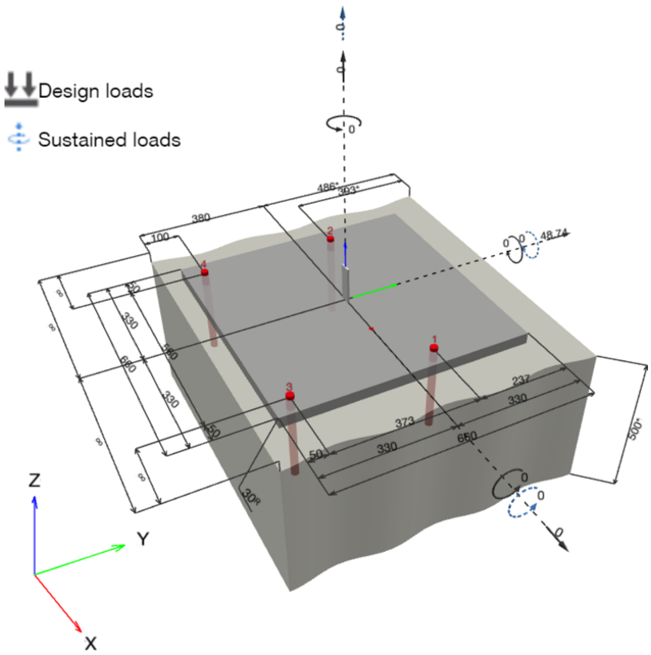
Specifier's comments:

1 Input data

Anchor type and size:	**HIT-RE 500 V4 + HAS-U 5.8 M20**
Return period (service life in years):	50
Item number:	2223877 HAS-U 5.8 M20x300 (insert) / 2287552 HIT-RE 500 V4 (mortar)
Effective embedment depth:	$h_{ef,act}$ = 210.0 mm ($h_{ef,limit}$ = - mm)
Material:	5.8
Approval No.:	ETA 20/0541
Issued I Valid:	04/09/2021 | -
Proof:	Design Method EN 1992-4, Chemical
Stand-off installation:	e_b = 0.0 mm (no stand-off); t = 30.0 mm
Baseplate^R :	l_x x l_y x t = 660.0 mm x 660.0 mm x 30.0 mm; (Recommended plate thickness: not calculated)
Profile:	Flat bar, 30 x 5,0; (L x W x T) = 30.0 mm x 5.0 mm
Base material:	cracked concrete, C30/37, $f_{c,cyl}$ = 30.00 N/mm^2; h = 500.0 mm, Temp. short/long: 20/0 °C, User-defined partial material safety factor γ_c = 1.500
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	No reinforcement or Reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)
	no longitudinal edge reinforcement
	Reinforcement to control splitting acc. to EN 1992-4, 7.2.1.7 (2) b) 2) present

^{R} - The anchor calculation is based on a rigid baseplate assumption.

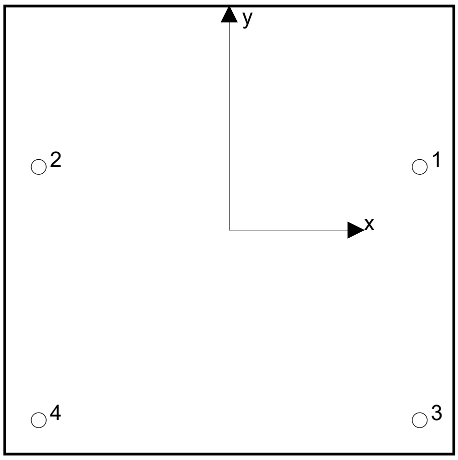
Geometry [mm] & Loading [kN, kNm]



1.1 Load combination

Case	Description	Forces [kN] / Moments [kNm]	Seismic	Fire	Max. Util. Anchor [%]
1	Combination 1	$N = 0.000$; $V_x = 0.000$; $V_y = 48.740$; $M_x = 0.000$; $M_y = 0.000$; $M_z = 0.000$; $N_{sus} = 0.000$; $M_{x,sus} = 0.000$; $M_{y,sus} = 0.000$;	no	no	45

2 Load case/Resulting anchor forces



Anchor reactions [kN]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	12.185	0.000	12.185
2	0.000	12.185	0.000	12.185
3	0.000	12.185	0.000	12.185
4	0.000	12.185	0.000	12.185

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [N/mm^2]
resulting tension force in (x/y)=(0.0/0.0): 0.000 [kN]
resulting compression force in (x/y)=(0.0/0.0): 0.000 [kN]

Anchor forces are calculated based on the assumption of a rigid baseplate.

3 Tension load ((EN 1992-4, Section 7.2.1))

	Load [kN]	Capacity [kN]	Utilization β_N [%]	Status
Steel failure*	N/A	N/A	N/A	N/A
Concrete Breakout failure**	N/A	N/A	N/A	N/A
Splitting failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

4 Shear load ((EN 1992-4, Section 7.2.2))

	Load [kN]	Capacity [kN]	Utilization β_V [%]	Status
Steel failure (without lever arm)*	12.185	58.848	21	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout failure**	48.740	321.520	16	OK
Concrete edge failure in direction y+**	48.740	109.811	45	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel failure (without lever arm)

$V_{Ed} \leq V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{M,s}}$ EN 1992-4, Table 7.2

$V_{Rk,s} = k_7 \cdot V^0_{Rk,s}$ EN 1992-4, Eq. (7.35)

$V^0_{Rk,s}$ [kN]	k_7	$V_{Rk,s}$ [kN]	$\gamma_{M,s}$	$V_{Rd,s}$ [kN]	V_{Ed} [kN]
73.560	1.000	73.560	1.250	58.848	12.185

4.2 Pryout failure (concrete cone relevant)

$V_{Ed} \leq V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{M,c,p}}$ EN 1992-4, Table 7.2

$V_{Rk,cp} = k_8 \cdot \min\{N_{Rk,c}; N_{Rk,p}\}$ EN 1992-4, Eq. (7.39c)

$N_{Rk,c} = N^0_{Rk,c} \cdot \frac{A_{c,N}}{A^0_{c,N}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec1,N} \cdot \psi_{ec2,N} \cdot \psi_{M,N}$ EN 1992-4, Eq. (7.1)

$N^0_{Rk,c} = k_1 \cdot \sqrt{f_{ck}} \cdot h_{ef}^{1,5}$ EN 1992-4, Eq. (7.2)

$A^0_{c,N} = s_{cr,N} \cdot s_{cr,N}$ EN 1992-4, Eq. (7.3)

$\psi_{s,N} = 0.7 + 0.3 \cdot \frac{c}{c_{cr,N}} \leq 1.00$ EN 1992-4, Eq. (7.4)

$\psi_{ec1,N} = \frac{1}{1 + \left(\frac{2 \cdot e_{V,1}}{s_{cr,N}}\right)} \leq 1.00$ EN 1992-4, Eq. (7.6)

$\psi_{ec2,N} = \frac{1}{1 + \left(\frac{2 \cdot e_{V,2}}{s_{cr,N}}\right)} \leq 1.00$ EN 1992-4, Eq. (7.6)

$\psi_{M,N} = 1$ EN 1992-4, Eq. (7.7)

$A_{c,N}$ [mm^2]	$A^0_{c,N}$ [mm^2]	$c_{cr,N}$ [mm]	$s_{cr,N}$ [mm]	k_8	$f_{c,cyl}$ [N/mm^2]
937,720	396,900	315.0	630.0	2.000	30.00

$e_{c1,V}$ [mm]	$\psi_{ec1,N}$	$e_{c2,V}$ [mm]	$\psi_{ec2,N}$	$\psi_{s,N}$	$\psi_{re,N}$	$\psi_{M,N}$
0.0	1.000	0.0	1.000	0.795	1.000	1.000

k_1	$N^0_{Rk,c}$ [kN]	$\gamma_{M,c,p}$	$V_{Rd,cp}$ [kN]	V_{Ed} [kN]
7.700	128.345	1.500	321.520	48.740

Group anchor ID
1-4

4.3 Concrete edge failure in direction y+

$V_{Ed} \leq V_{Rd,c} = \dfrac{V_{Rk,c}}{\gamma_{M,c}}$	EN 1992-4, Table 7.2
$V_{Rk,c} = k_T \cdot V^0_{Rk,c} \cdot \dfrac{A_{c,V}}{A^0_{c,V}} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$	EN 1992-4, Eq. (7.40)
$V^0_{Rk,c} = k_9 \cdot d^{\alpha}_{nom} \cdot l^{\beta}_f \cdot \sqrt{f_{ck}} \cdot c_1^{1,5}$	EN 1992-4, Eq. (7.41)
$\alpha = 0.1 \cdot \left(\dfrac{l_f}{c_1}\right)^{0,5}$	EN 1992-4, Eq. (7.42)
$\beta = 0.1 \cdot \left(\dfrac{d_{nom}}{c_1}\right)^{0,2}$	EN 1992-4, Eq. (7.43)
$A^0_{c,V} = 4.5 \cdot c_1^2$	EN 1992-4, Eq. (7.44)
$\psi_{s,V} = 0.7 + 0.3 \cdot \dfrac{c_2}{1.5 \cdot c_1} \leq 1.00$	EN 1992-4, Eq. (7.45)
$\psi_{h,V} = \left(\dfrac{1.5 \cdot c_1}{h}\right)^{0,5} \geq 1.00$	EN 1992-4, Eq. (7.46)
$\psi_{ec,V} = \dfrac{1}{1 + \left(\dfrac{2 \cdot e_V}{3 \cdot c_1}\right)} \leq 1.00$	EN 1992-4, Eq. (7.47)
$\psi_{\alpha,V} = \sqrt{\dfrac{1}{(\cos \alpha_V)^2 + (0.5 \cdot \sin \alpha_V)^2}} \geq 1.00$	EN 1992-4, Eq. (7.48)

l_f [mm]	d_{nom} [mm]	k_9	α	β	$f_{c,cyl}$ [N/mm^2]
210.0	20.00	1.700	0.073	0.055	30.00

c_1 [mm]	$A_{c,V}$ [mm^2]	$A^0_{c,V}$ [mm^2]
393.0	869,500	695,020

$\psi_{s,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	$e_{c,V}$ [mm]	$\psi_{ec,V}$	$\psi_{re,V}$
1.000	1.086	1.000	0.0	1.000	1.000

$V^0_{Rk,c}$ [kN]	k_T	$\gamma_{M,c}$	$V_{Rd,c}$ [kN]	V_{Ed} [kN]
121.258	1.0	1.500	109.811	48.740

5 Displacements (highest loaded anchor)

Short term loading:

N_{Sk} =	0.000 [kN]	δ_N =	0.0000 [mm]
V_{Sk} =	18.052 [kN]	δ_V =	0.7221 [mm]
		δ_{NV} =	0.7221 [mm]

Long term loading:

N_{Sk} =	0.000 [kN]	δ_N =	0.0000 [mm]
V_{Sk} =	18.052 [kN]	δ_V =	1.0831 [mm]
		δ_{NV} =	1.0831 [mm]

Comments: Tension displacements are valid with half of the required installation torque moment for uncracked concrete! Shear displacements are valid without friction between the concrete and the baseplate! The gap due to the drilled hole and clearance hole tolerances are not included in this calculation!

The acceptable anchor displacements depend on the fastened construction and must be defined by the designer!

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid baseplates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the baseplate are not considered - the baseplate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required baseplate thickness with CBFEM to limit the stress of the baseplate based on the assumptions explained above. The proof if the rigid baseplate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Checking the transfer of loads into the base material is required in accordance with EN 1992-4, Annex A!
- The design is only valid if the clearance hole in the fixture is not larger than the value given in Table 6.1 of EN 1992-4! For larger diameters of the clearance hole see section 6.2.2 of EN 1992-4!
- The accessory list in this report is for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- For the determination of the $\psi_{re,v}$ (concrete edge failure) the minimum concrete cover defined in the design settings is used as the concrete cover of the edge reinforcement.
- Characteristic bond resistances depend on short- and long-term temperatures.
- Edge reinforcement is not required to avoid splitting failure
- Load transfer from supplementary reinforcement to the structural member shall be verified by the responsible structural engineer.
- With supplementary reinforcement and post-installed anchors, please ensure that in the jobsite the rebars are not drilled through.
- The characteristic bond resistances depend on the return period (service life in years): 50

Fastening meets the design criteria!

7 Installation data

Baseplate, steel: S 355; E = 210,000.00 N/mm^2; f_{yk} = 355.00 N/mm^2

Profile: Flat bar, 30 x 5,0; (L x W x T) = 30.0 mm x 5.0 mm

Hole diameter in the fixture: d_f = 22.0 mm

Plate thickness (input): 30.0 mm

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and size: HIT-RE 500 V4 + HAS-U 5.8 M20

Item number: 2223877 HAS-U 5.8 M20x300 (insert) / 2287552 HIT-RE 500 V4 (mortar)

Maximum installation torque: 150 Nm

Hole diameter in the base material: 22.0 mm

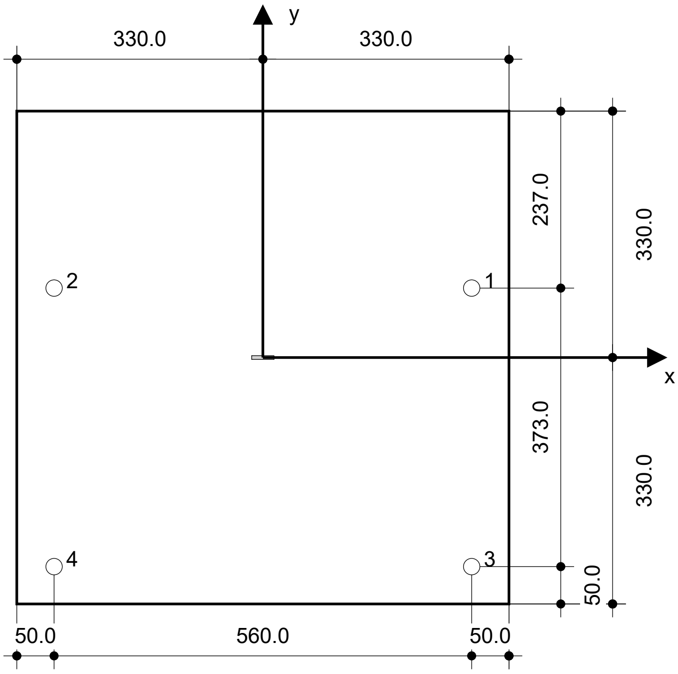
Hole depth in the base material: 210.0 mm

Minimum thickness of the base material: 254.0 mm

Hilti HAS-U threaded rod with HIT-RE 500 V4 injection mortar with 210 mm embedment h_ef, M20, Steel galvanized, Hammer drilling installation per ETA 20/0541

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor [mm]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	280.0	93.0	-	-	473.0	393.0
2	-280.0	93.0	-	-	473.0	393.0
3	280.0	-280.0	-	-	100.0	766.0
4	-280.0	-280.0	-	-	100.0	766.0

8 Remarks; Your Cooperation Duties

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Approval of Temporary Works Design

Our Client:	
Project Name:	
Principle Contractor:	

We trust the temporary works design meets all the requirements for this project. To confirm this and in order for us to proceed with the manufacturing and delivery phase, please read and put your signature to the statement below.

As the signatory below, I confirm that this temporary works design has been reviewed and has been approved as meeting all criteria and requirements set out for this project.

On behalf of: - Client Name

Name:	
Position in the company:	
Signed:	
Date:	

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