3.0 Design Code, Reference and Data

3.1 Codes & Design References

- Code of Practice on Wind Effects in Hong Kong 2004 (CoP WE 2004)
- Explanatory Materials to the Code of Practice on Wind Effects 2004 (EM CoP WE 2004)
- Code of Practice for the Structural Use of Steel 2011 (CoP SUOS 2011)
- BS EN 40-3-1:2013 – Lighting Columns Part 3-1: Specification for characteristic loads
- BS EN 40-3-3:2013 – Lighting Columns Part 3-3: Verification by calculation
- BS EN 1991-1.4-2005 (2010) Wind Actions
- Guidelines for Converting between Various Wind Averaging Periods in Tropical Cyclone Conditions (WMO/TD-No 1555)

3.2 Load Cases

- *Dead Load (DL)*

Unit weight of steel	= 77 kN/m^3
Lantern (as advised by the supplier) – 12kg	= 0.12kN
CCTV (as advised by the supplier) – 3.07 kg	= 0.05 kN
Control box (as advised by the supplier) – 12 kg	= 0.12 kN

- *Imposed Load (LL)*

  No imposed load has been considered.

- *Wind Load (WL)*

  Wind load calculation is based on the following equation:

$$F_c = A_c \times c \times q_{(Z)}$$

where

F_c is the partial horizontal force due to wind pressure acting at the centre of shaft;

A_c is the projected area on the vertical plane normal to direction of the wind;

c is the shape coefficient for the mast or column shaft;

$q_{(Z)}$ is the characteristic wind pressure at a height z in metres above ground level.

As for the characteristic wind pressure $q_{(Z)}$, it has been derived based on the following equations in BS EN 40-3-1 using a modified wind velocity profile (ie, the 10-minute mean wind velocity) based on the 3-second gust wind velocity profile in EM CoP WE 2004.

$$q_{(Z)} = \delta \times \beta \times f \times Ce_{(Z)} \times q_{(10)}$$

where

δ is a factor depending on the column size, taken as $\delta = 1 - 0.01h$ where h is pole height

β is a factor depending on the dynamic behaviour of the lighting column = 1.0024 - $0.005T^4 - 0.22793T^3 + 0.67262T$ and T is the 1st mode period of mast (See model in **Appendix B**)

	4m Single Arm	4m Double Arm	8m Single Arm	8m Double Arm	CCTV column
Height (m)	4	4	8	8	6
δ	0.96	0.96	0.92	0.92	0.94
Period (s)	0.13	0.13	0.54	0.55	0.2
β	1.086	1.086	1.307	1.311	1.14

f is a topography factor which shall be taken as **1.0** (as the mast and lighting columns are erected on flat ground not located at topographical significant zone);

$Ce_{(Z)}$ is a factor depending on the terrain of the site and the height above ground. This term shall be taken as **1.0** since the terrain effects in Hong Kong (ie, open sea condition) is already included in the wind velocity profile;

Table 2 — Exposure coefficient $c_e(z)$

Height above ground Z (m)	Terrain category			
	I	II	III	IV
20	3,21	2,81	2,28	1,72
19	3,17	2,77	2,24	1,69
18	3,14	2,74	2,20	1,65
17	3,10	2,70	2,16	1,60
16	3,07	2,66	2,11	1,56
15	3,03	2,62	2,07	1,56
14	2,98	2,57	2,02	1,56
13	2,94	2,52	1,96	1,56
12	2,89	2,47	1,91	1,56
11	2,83	2,41	1,85	1,56
10	2,78	2,35	1,78	1,56
9	2,71	2,29	1,71	1,56
8	2,64	2,21	1,63	1,56
7	2,57	2,13	1,63	1,56
6	2,48	2,04	1,63	1,56
5	2,37	1,93	1,63	1,56
4	2,25	1,80	1,63	1,56
3	2,09	1,80	1,63	1,56
2	1,88	1,80	1,63	1,56
1	1,88	1,80	1,63	1,56

Fig. 3 - Table 2 in BS EN 40-3-1

For conservative, take $Ce_{(Z)}$ will be taken from Fig. 3.

	4m Single Arm	4m Double Arm	8m Single Arm	8m Double Arm	CCTV Col.
Height (m)	4	4	8	8	6
$Ce_{(Z)}$	2.25	2.25	2.64	2.64	2.48

$q_{(10)}$ is the reference wind pressure;

$$q_{(10)} = 0.5 \times \rho \times (C_s)^2 \times (V_{ref})^2$$

ρ is the density of air which shall be taken as **1.2kg/m³**, as in EM CoP WE 2004;

C_s is a statistical factor related to the annual probability of exceedance which shall be taken as **1.0** since V_{ref} in Hong Kong is based on a mean return period of 50 years;

V_{ref} is the 10-minute mean wind velocity (see Table 1). V_{ref} is converted from the 3-second gust wind velocity in EM CoP WE 2004.

Table 1 – Recommended wind speed conversion factors for tropical cyclone conditions

Exposure at +10 m		Reference Period	Gust Factor G_{τ,T_0}				
			Gust Duration τ (s)				
Class	Description	T_0 (s)	3	60	120	180	600
In-Land	Roughly open terrain	3600	1.75	1.28	1.19	1.15	1.08
		600	1.66	1.21	1.12	1.09	1.00
		180	1.58	1.15	1.07	1.00	
		120	1.55	1.13	1.00		
		60	1.49	1.00			
Off-Land	Offshore winds at a coastline	3600	1.60	1.22	1.15	1.12	1.06
		600	1.52	1.16	1.09	1.06	1.00
		180	1.44	1.10	1.04	1.00	
		120	1.42	1.08	1.00		
		60	1.36	1.00			
Off-Sea	Onshore winds at a coastline	3600	1.45	1.17	1.11	1.09	1.05
		600	1.38	1.11	1.05	1.03	1.00
		180	1.31	1.05	1.00	1.00	
		120	1.28	1.03	1.00		
		60	1.23	1.00			
At-Sea	> 20 km offshore	3600	1.30	1.11	1.07	1.06	1.03
		600	1.23	1.05	1.02	1.00	1.00
		180	1.17	1.00	1.00	1.00	
		120	1.15	1.00	1.00		
		60	1.11	1.00			

$$\frac{v_{3\,sec}}{mean\ wind\ speed} = 1.45$$

$$\frac{v_{600\,sec}}{mean\ wind\ speed} = 1.05$$

$$\frac{v_{600\,sec}}{v_{3\,sec}} = \frac{1.05}{1.45}$$

$$v_{600sec} = 0.724\ v_{3\,sec}$$

Refer Clause 1.21 in EM CoP WE 2004, design 3 second gust wind speed v_z at height z is calculated by the following equations

$$v_z = \overline{v_z}(1 + 3.7I_z);\ \ \overline{v_z} = 1.05\overline{v_g}\left(\frac{z}{z_g}\right)^{\propto};\ I_z = I_g\left(\frac{z}{z_g}\right)^{-\propto}$$

where

$\overline{v_g}$ is hourly mean wind speed at gradient height = 56.6 m/s

I_g is turbulence intensity at gradient height = 0.087

z_g is gradient height = 500m

I_g	is turbulence intensity at gradient height	= 0.087
$\propto$	is power exponent for mean wind	= 0.11

	4m Single Arm	4m Double Arm	8m Single Arm	8m Double Arm	CCTV Column
Height (m)	4	4	8	8	6
$v_{3\,sec}$ (m/s)	54.1	54.1	56.8	56.8	55.7
v_{ref} (m/s)	39.2	39.2	41.2	41.2	40.3
$q_{(10)}$ (kPa)	0.92	0.92	1.017	1.017	0.975
$q_{(Z)}$ (kPa)	2.158	2.158	3.226	3.238	2.591

Adopt characteristic wind pressure $q_{(Z)} = \mathbf{3.84\ kPa}$

ie. equivalent wind speed $v = \sqrt{\frac{q_z}{0.5\,\rho}} = \sqrt{\frac{3.84}{0.5*1.2}} = \mathbf{80\ m/s}$

- *Wind coefficients c*

For the mast is 12-sided regular polygon, as per Cl. 5.3.3 in BS EN 40-3-1, the shape coefficient shall be taken from EN 1991-1-4. In accordance with Table 7.11 in Fig. 3, the wind coefficient for regular polygonal sections is related to Reynolds number R_e,

$$R_e = \frac{V\,D}{v}$$

where

V is wind speed = 80 m/s

v is kinematic viscosity of air = 15 x 10^{-6} m^2/s

D is diameter of circumscribed circumference of mast

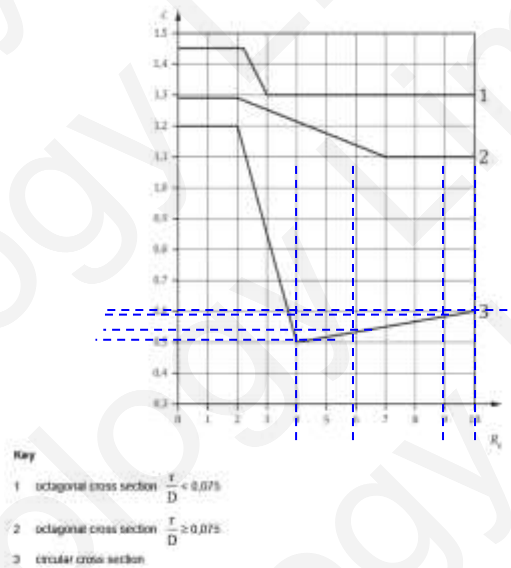


Figure 3 — Shape coefficient for circular and octagonal cross-sections

	4m Single Arm	4m Double Arm	8m Single Arm	8m Double Arm	CCTV column
Height (m)	4	4	8	8	6
V (m/s)	80	80	80	80	80
D1 (m)	76	76	114	114	114
R_e	$4.1 \cdot 10^5$	$4.1 \cdot 10^5$	$6.1 \cdot 10^5$	$6.1 \cdot 10^5$	$6.1 \cdot 10^5$
$c1$	0.5	0.5	0.53	0.53	0.53
D2 (m)	165	165	194	194	194
R_e	$8.8 \cdot 10^5$	$8.8 \cdot 10^5$	$10.4 \cdot 10^5$	$10.4 \cdot 10^5$	$10.4 \cdot 10^5$
$c2$	0.58	0.58	0.6	0.6	0.6

Adopt Wind coefficient $c = \mathbf{1.2}$ for ULS design
Adopt Wind coefficient $c = \mathbf{0.75}$ for SLS design

For the lanterns, use wind coefficient c = **1.0** (as per Clause 6.3.4 in BS EN 40-3-3)
For CCTV, use wind coefficient c = **2.0** (as open frame element in CoP WE 2004)

Therefore, design wind pressures:
i) on the llighting columns WL = 1.2 x 3.84 (ULS)
WL = 0.75 x 3.84 (SLS)

ii) on lanterns WL = 1.0 x 3.84

iii) on the mast WL = 1.2 x 3.84 (ULS)
WL = 0.75 x 3.84 (SLS)

iv) on CCTV WL = 2.0 x 3.84

3.3 *Load Combination*

- Ultimate limit state (ULS):
  1.4DL (Adverse) + 1.4WL ; 1.0 DL (Beneficial) + 1.4 WL
- Serviceability limit state (SLS):
  1.0 DL +1.0 WL

3.4 *Deflection criteria*

As per Cl. 6.5.1 in BS EN 40-3-3, the most stringent maximum horizontal deflection, Δ

$\Delta \leq 0.04\ (h + w)$ = 0.04 x 16 = 0.64m = 640mm

where

h is the nominal height of the lighting columns, in metres

w is the arm (or called bracket) projection, in metres.

	4m Single Arm	4m Double Arm	8m Single Arm	8m Double Arm	CCTV Column
Height h (m)	4	4	8	8	6
Bracket w (m)	0	0.35	0	0.35	0
Δ (mm)	160	174	320	334	240
Max. deflection	64	102	304	329	154
	Okay	Okay	Okay	Okay	Okay

The horizontal deflection lighting columns and mast i analyzed by computer model, using the wind forces developed in section 2.0. Resultant deflection is checked within the above limit.

3.5 *Materials*

- *Structural Steel*

Pole	Section	Steel Grade	Design Strength fy (N/mm²)
8m lighting Column & 6m CCTV Mast	114 x 6 CHS, 194 x 6 CHS, 6mm thk reinforcing flat bars	Gr 65	450
4m lighting Column	76 x 4 CHS, 165 x 4 CHS, 4mm thk reinforcing flat bars	Q345C	310

Brackets	60 x 3 CHS	Q345C	310
Others	Plates	Q345C	310

Modulus of elasticity E = 205,000 N/mm^2

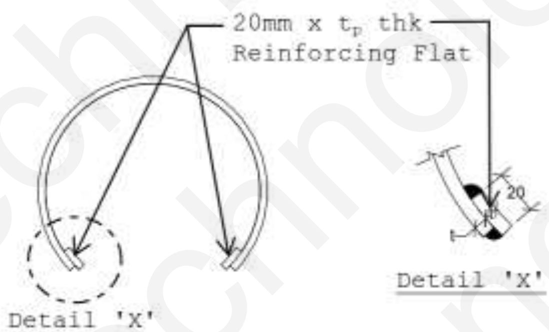
For class 1 steet, material factor g_{m1} is taken as 1.0 for the structural steel yield strength (Ref: Table 4.1, CoP SUOS 2011).

- *Fillet Weld*

  Electrode classification E50. Design strength p_w = 200 N/mm^2

- *Door Reinforcing*

At the access opening on base portion of the columns, 2 reinforcing flat bars are to be welded for strengthening. The reinforced opening is designed in accordance with clause 5.6.2.2 in BS EN 40-3-3:2013. Detail is shown as follows. Detail checking is presented in section 4-6.



Door Reinforcing Detail

Remarks: t_p shall be the thickness of column

4.0 Design for Lighting Columns

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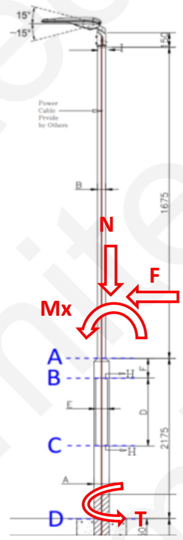
4m Single Arm Lighting Column



Column Height	h	=	4	m	
Shaft Dia.	B	=	0.076	m	
Thickness	t_s	=	4	mm	
Weight	w_s	=	0.07	kN/m	
Length	L_s	=	1.825	m	
Base Dia.	A	=	0.165	m	
Thickness	t_b	=	4	mm	
Weight	w_b	=	0.16	kN/m	
Length	L_b	=	2.175	m	
Bracket Dia.	D_p	=	0.0	m	
Thickness	t_p	=	0.0	mm	
Weight	w_p	=	0.00	kN/m	
Length	P	=	0.0	m	

Door Opening

Door Height	D	=	1.000	m
Door Width	E	=	0.14	m
Distance	F	=	0.275	m

Lantern

Size $b \times d \times h$ = 0.763 x 0.352 x 0.167

Inclination θ = 15 deg

Ht above top h' = $h + b \sin\theta$ = 0.364 m

Frontal Area A_{LX} = $b \times h$ = 0.13 m^2

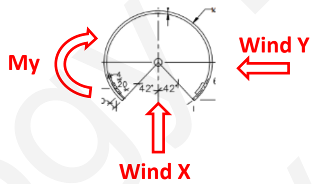
A_{LY} = $d \times h$ = 0.06 m^2

Weight W_L = 0.12 kN

Wind pressure q = 3.84 kPa

Coefficients c_1 = 1.2 (for circular section)

c_2 = 1 (for lantern)



Dead Load at each section location

$N_A = W_L + w_p \times P + w_s \times L_s$ = 0.25 kN

$N_B = N_A + w_b \times F$ = 0.29 kN

$N_C = N_B + w_b \times D$ = 0.45 kN

$N_D = N_C + w_b \times (L_b - D - F)$ = 0.59 kN

Eccentric Moment

$M_{DLY} = W_L(b/2+B/2) + w_p P(P/2+B/2)$

= 0.05 kNm

Wind X case

Wind Shear F_X at each section location

$F_A = q(c_2 A_L + c_1 D_p P + c_1 B L_s)$ = 0.49 + 0.00 + 0.64 = 1.13 kN

$F_B = F_A + q(c_1 A F)$ = 1.13 + 0.21 = 1.34 kN

$F_C = F_B + q(c_1 A D)$ = 1.34 + 0.76 = 2.10 kN

$F_D = F_C + q(c_1 A (L_b - D - F))$ = 2.10 + 0.68 = 2.78 kN

Wind Moment M_Y at each section location

$M_A = q(c_2 A_{LX}(L_s + h) + c_1 D_p P L_s + c_1 B L_s^2/2)$ = 1.65 kNm

$M_B = M_A + F_A F + q(c_1 A F^2/2)$ = 1.99 kNm

$M_C = M_B + F_B D + q(c_1 A D^2/2)$ = 3.71 kNm

$M_D = M_C + F_C (L_b - D - F) + q(c_1 A (L_b - D - F)^2/2)$ = 5.91 kNm

Torsional Moment due to Wind

Torsion due to wind on lantern & bracket at Centreline of column

$T_1 = q(c_2 A_{LX}(b/2+P+B/2) + c_1 D_p P(P/2+B/2))$ = 0.21 kNm

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4m Single Arm Lighting Column

Wind Y case

Wind Shear F_Y at each section location

$F_A = q(\ c_2 A_{LY} + c_1 B L_s)$ = 0.23 + 0.64 = 0.86 kN

$F_B = F_A + q(\ c_1 A\ F)$ = 0.86 + 0.21 = 1.07 kN

$F_C = F_B + q(\ c_1 A\ D)$ = 1.07 + 0.76 = 1.83 kN

$F_D = F_C + q(\ c_1 A\ (L_b\text{-}D\text{-}F))$ = 1.83 + 0.68 = 2.52 kN

Wind Moment M_X at each section location

$M_A = q(\ c_2 A_{LY}\ (L_s + h') + c_1 B L_s^2/2)$ = 1.08 kNm

$M_B = M_A + F_A\ F + q(\ c_1 A\ F^2/2)$ = 1.34 kNm

$M_C = M_B + F_B\ D + q(\ c_1 A\ D^2/2)$ = 2.80 kNm

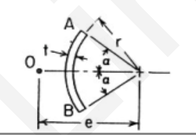
$M_D = M_C + F_C\ (L_b\text{-}D\text{-}F) + q(\ c_1 A\ (L_b\text{-}D\text{-}F)^2/2)$ = 4.76 kNm

Torsion due to eccentricity of shear centres at door opening

$e = 2\,r\,(\sin\alpha - \alpha\cos\alpha)/(\alpha - \sin\alpha\,\cos\alpha)$ = 0.13 m

α = 132 °

9. Segment of a circular tube



$$e = 2r\frac{\sin\alpha - \alpha\cos\alpha}{\alpha - \sin\alpha\cos\alpha}$$

ie. Torsion at section location

$T_B = F_B\ e$ = 1.34 x 0.13 = 0.18 kNm

$T_C = F_C\ e$ = 2.10 x 0.13 = 0.28 kNm

Summary of axial, shear, moments and torsion at each section location

	Case 1: DL + WLX					Case 2: DL + WLY				
Sec.	**Axial**	**Shear**	**Moment**	**Moment**	**Torsion**	**Axial**	**Shear**	**Moment**	**Moment**	**Torsion**
	N (kN)	**F (kN)**	**M_X (kNm)**	**M_Y (kNm)**	**T (kNm)**	**N (kN)**	**F (kN)**	**M_X (kNm)**	**M_Y (kNm)**	**T (kNm)**
A	0.25	1.13	0.05	1.65	0.21	0.25	0.86	1.13	0.00	0.00
B	0.29	1.34	0.05	1.99	0.21	0.29	1.07	1.39	0.00	0.18
C	0.45	2.10	0.05	3.71	0.21	0.45	1.83	2.85	0.00	0.28
D	0.59	2.78	0.05	5.91	0.21	0.59	2.52	4.81	0.00	0.00

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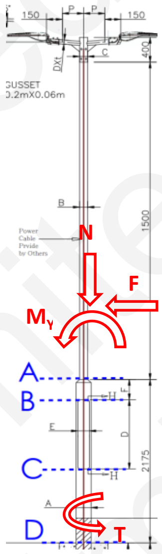
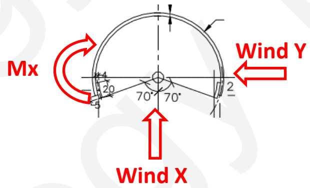
4m Double Arm Lighting Column



Column Height h = 4 m
Shaft Dia. B = 0.076 m
Thickness t_s = 4 mm
Weight w_s = 0.07 kN/m
Length L_s = 1.825 m
Base Dia. A = 0.165 m
Thickness t_b = 4 mm
Weight w_b = 0.16 kN/m
Length L_b = 2.175 m
Bracket Dia. D_p = 0.06 m
Thickness t_p = 3.0 mm
Weight w_p = 0.04 kN/m
Length P = 0.35 m

Door Opening
Door Height D = 0.700 m
Door Width E = 0.11 m
Distance F = 0.205 m

Lantern (2 nos)
Size b x d x h = 0.763 x 0.352 x 0.167
Inclination θ = 5 deg
Ht above top h' = $h + b \sin\theta$ = 0.233 m
Frontal Area A_{Lx} = b x h = 0.13 m^2
A_{LY} = d x h = 0.06 m^2
Weight W_L = 0.12 kN



Wind pressure q = 3.84 kPa
Coefficients c_1 = 1.2 (for circular section)
c_2 = 1 (for lantern)

Dead Load at each section location
$N_A = 2(W_L + w_p \times P) + w_s \times L_s$ = 0.40 kN
$N_B = N_A + w_b \times F$ = 0.43 kN
$N_C = N_B + w_b \times D$ = 0.54 kN
$N_D = N_C + w_b \times (L_b - D - F)$ = 0.73 kN

Eccentric Moment
$M_{DLY} = 2\,[\,W_L(b/2+B/2) + w_pP(P/2+B/2)\,]$
= 0.11 kNm

Wind X case
Wind Shear F_X at each section location
$F_A = q(\ 2c_2 A_L + 2c_1 D_p P + c_1 B L_s)$ = 0.98 + 0.19 + 0.64 = 1.81 kN
$F_B = F_A + q(\ c_1 A\, F)$ = 1.81 + 0.16 = 1.97 kN
$F_C = F_B + q(\ c_1 A\, D)$ = 1.97 + 0.53 = 2.50 kN
$F_D = F_C + q(\ c_1 A\,(L_b - D - F))$ = 2.50 + 0.97 = 3.46 kN

Wind Moment M_Y at each section location
$M_A = q(\ 2c_2 A_L (L_s + h') + 2c_1 D_p P L_s + c_1 B L_s^2/2)$ = 2.95 kNm
$M_B = M_A + F_A\ F + q(\ c_1 A\, F^2/2)$ = 3.34 kNm
$M_C = M_B + F_B\ D + q(\ c_1 A\, D^2/2)$ = 4.90 kNm
$M_D = M_C + F_C\ (L_b - D - F) + q(\ c_1 A\,(L_b - D - F)^2/2)$ = 8.69 kNm

Torsional Moment due to Wind
Torsion due to wind on lantern & bracket at Centreline of column (assume only ONE end with lantern)
$T_1 = q(\ c_2 A_L (b/2+P+B/2) + c_1 D_p P(P/2+B/2))$ = 0.40 kNm

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4m Double Arm Lighting Column

Wind Y case

Wind Shear F_Y at each section location

$F_A = q(\ c_2 A_{LY} + c_1 B L_s)$ = 0.23 + 0.64 = 0.86 kN

$F_B = F_A + q(\ c_1 A\ F)$ = 0.86 + 0.16 = 1.02 kN

$F_C = F_B + q(\ c_1 A\ D)$ = 1.02 + 0.53 = 1.55 kN

$F_D = F_C + q(\ c_1 A\ (L_b\text{-}D\text{-}F))$ = 1.55 + 0.97 = 2.52 kN

Wind Moment M_X at each section location

$M_A = q(\ c_2 A_{LY}\ (L_s+h') + c_1 B L_s^2/2)$ = 1.05 kNm

$M_B = M_A + F_A\ F + q(\ c_1 A\ F^2/2)$ = 1.24 kNm

$M_C = M_B + F_B\ D + q(\ c_1 A\ D^2/2)$ = 2.14 kNm

$M_D = M_C + F_C\ (L_b\text{-}D\text{-}F) + q(\ c_1 A\ (L_b\text{-}D\text{-}F)^2/2)$ = 4.73 kNm

Torsion due to eccentricity of shear centres at door opening

$e = 2r(\sin\alpha - \alpha\cos\alpha)/(\alpha - \sin\alpha\cos\alpha)$ = 0.11 m

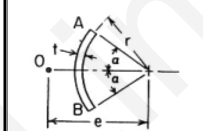
α = 110 °

ie. Torsion at section location

$T_B = F_B\ e$ = 1.97 x 0.11 = 0.23 kNm

$T_C = F_C\ e$ = 2.50 x 0.11 = 0.29 kNm

9. Segment of a circular tube



$$e = 2r\frac{\sin\alpha - \alpha\cos\alpha}{\alpha - \sin\alpha\cos\alpha}$$

Summary of axial, shear, moments and torsion at each section location

	Case 1: DL + WLX					Case 2: DL + WLY				
Sec.	**Axial N (kN)**	**Shear F (kN)**	**Moment M_X (kNm)**	**Moment M_Y (kNm)**	**Torsion T (kNm)**	**Axial N (kN)**	**Shear F (kN)**	**Moment M_X (kNm)**	**Moment M_Y (kNm)**	**Torsion T (kNm)**
A	0.40	1.81	0.11	2.95	0.40	0.40	0.86	1.15	0.00	0.00
B	0.43	1.97	0.11	3.34	0.40	0.43	1.02	1.35	0.00	0.23
C	0.54	2.50	0.11	4.90	0.40	0.54	1.55	2.25	0.00	0.29
D	0.73	3.46	0.11	8.69	0.40	0.73	2.52	4.83	0.00	0.00

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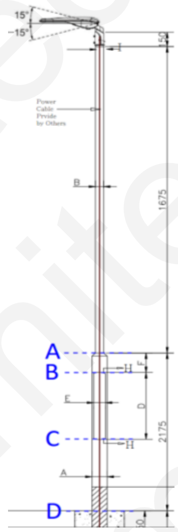
Design for 4m Lighting Columns (Single and Double Arm)

Calculation of Section Capacities



Section "A" - Interface of shaft and base

Diameter	D	=	76.0	mm	Refer Table 7.2, CoP Steel 2011
Wall thickness	t	=	4	mm	D/t = 19.0 < 40 ε^2
Mean radius	R	=	36.0	mm	Class 1 plastic
Section Area	A	=	9.0	cm^2	
Elastic Modulus	Z_p	=	$4R^2t$ = 20.7	cm^3	(Closed circular section)
2nd Moment of inertia	I	=	58.8	cm^4	
Radius of gyration	r	=	2.55	cm	
Design Strength	f_y	=	310	N/mm^2	
Modulus of Elasticity	E	=	205	kN/mm^2	
Partial Material Factor	γ_m	=	1.00		

a) Bending Moment of Resistance

$M_{ux} = M_{uy} = M_{up} = f_y \Phi_1 Z_p / 10^3 \gamma_m$ (Refer Cl. 5.6.2.1, BS EN40-3-3:2013)

$\xi = (R/t)\,\mathrm{sqrt}(f_y/E)$

$= 0.35$

$\Phi_1 = 1.0$ (Refer Figure 2 in BS EN 40-3-3:2103)

$$M_{up} = \frac{310 \times 1.0 \times 20.7}{10^3 \times 1.00}$$

$= \mathbf{6.43}$ kNm

b) Torsional Moment of Resistance

$T_u = f_y \Phi_2 \pi R^2 t / 10^3 \gamma_m$ (Refer Cl. 5.6.2.1, BS EN40-3-3:2013)

$\Phi_2 = \min.\{\ 0.474\,E / f_y\,(R/t)^{1.5}\ ;\ 1.0\ \}$

$= \min.\{\ 11.61\ ;\ 1.0\ \}$

$= 1.0$

$$T_u = \frac{310 \times 1.0\ \pi\ 36.0^2 \times 4.0\ /10^3}{10^3 \times 1.00}$$

$= \mathbf{5.05}$ kNm

c) Shear capacity (Refer Cl.8.2.1 , CoP Steel 2011)

$V_c = f_y\ (0.6\ A)\ /3^{0.5}$

$= 310.0 \times 5.4\ /3^{0.5}\ /\ 10 = \mathbf{97.2}$ kN

d) Compressive capacity

$P_c = p_c\ A_g$ (Refer Cl. 8.7.5, CoP Steel 2011)

$p_c = p_E p_y / [f_c + (f_c^2 - p_E p_y)^{0.5}]$ ($p_y = f_y$)

$l = L_E / r = 2 \times 182.5 / 2.55 = 143.2$

$p_E = \pi^2 E / l^2 = 98.7\ N/mm^2$

$\lambda_0 = 0.2(\pi^2 E/p_y)^{0.5} = 16.2\ N/mm^2$

$\alpha = 2.0$ (Curve a)

$h = \alpha(\lambda - \lambda_0)/1000 = 0.25$

$f_c = [p_y + (h+1)p_E] / 2 = 217\ N/mm^2$

$p_c = p_E p_y / [f_c + (f_c^2 - p_E p_y)^{0.5}] = 88.7\ N/mm^2$

$P_c = 88.7 \times 9.0\ /\ 10 = \mathbf{80.2}$ kN

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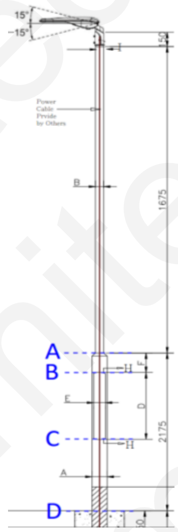
Design for 4m Lighting Columns (Single Arm Only)

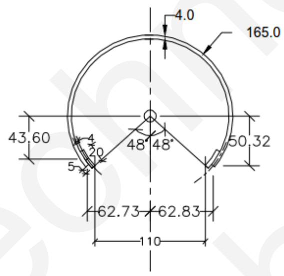
Calculation of Section Capacities



Section "B" & "C - Top & Bottom of Door Opening (with TYPE 4 Reinforcement)

Diameter	D	=	165	mm	Refer Table 7.2, CoP Steel 2011
Wall thickness	t	=	4	mm	D/t = 41.3 < 50 ε^2
Mean radius	R	=	80.5	mm	Class 2 compact
Door Opening					
Half angle of opening	θ	=	48	°	
Length	a	=	1000	mm	
Corner radius	N	=	20	mm	
Effective length	L	=	991	mm	L = a - 0.43N
Half length of straight edge	C	=	960	mm	C = a - 2N

TYPE 4 door reinforcement (Refer Cl. 5.6.2.2, BS EN40-3-3:2013)



Area: 1611.7035
Perimeter: 765.5102
Moments of inertia: X: 3298371.8288
Y: 6358880.4334
Radii of gyration: X: 45.2384
Y: 62.8127

Thickness	t_w	=	4	mm					
Width	d_w	=	20	mm					
Area	A_e	=	80	mm					
Section Modulus	Z_{pnr}	=	$FR^2 t\,[2\cos(\theta/2\text{-}90B_x/\pi) - \sin\theta + B_x\cos\theta]$				=	69.3	cm^3
	Z_{pyr}	=	$FR^2 t\,(1 + \cos\theta + B_y \sin\theta)$				=	95	cm^3
	B_x	=	A_e / Rt * m_{ox} / m_x = 0.25						
	B_y	=	A_e / Rt * m_{oy} / m_y = 0.22						
Ctr of door reinf. to x axis	m_{ox}	=	62.7	mm	Col. wall edge to x	m_x	=	62.8	mm
Ctr of door reinf. to y axis	m_{oy}	=	43.6	mm	Col. wall edge to y	m_y	=	50.3	mm

For circular sec.
F = 2.0

Section Area	A	=	16.1	cm^2
2nd Moment of inertia	I_x	=	329.8	cm^4
	I_y	=	635.9	cm^4
Radius of gyration	r_x	=	4.52	cm
	r_y	=	6.28	cm
Design Strength	f_y	=	310	N/mm^2
Modulus of Elasticity	E	=	205	kN/mm^2
Partial Material Factor	γ_m	=	1.00	

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a) Bending Moment of Resistance

$M_{ux} = f_y\, \Phi_6\, Z_{pnr} / 10^3 \gamma_m$ (Refer Cl. 5.6.2.2, BS EN40-3-3:2013)

$M_{uy} = f_y\, \Phi_6\, Z_{pyr} / 10^3 \gamma_m$

$\Phi_6 = (2t + t_w)^2 E / [(2t + t_w)^2 E + 0.32\, RLf_y] = 0.79 \le \Phi_1 = 1.0$

$$M_{ux} = \frac{310 \times 0.8 \times 69.3}{10^3 \times 1.00} = \mathbf{16.9}\ \text{kNm}$$

$$M_{uy} = \frac{310 \times 0.8 \times 94.8}{10^3 \times 1.00} = \mathbf{23.2}\ \text{kNm}$$

b) Torsional Moment of Resistance

$T_u = f_y\, \Phi_6 (\Phi_5 + P\, \Phi_7)\, R^3\, t / 10^3 \gamma_m L$ (Refer Cl. 5.6.2.1, BS EN40-3-3:2013)

$$\Phi_5 = \frac{10 \cos^2(\theta/2)}{1+1.73 \tan\theta} \left[\frac{1+2.15 \tan\theta + 0.85R/L}{1+2.15 \tan\theta + 0.85R/L + 3.8(R/L)^2} \right] = 2.84$$

$\Phi_7 = 4.4$ $R/L = 0.081$ (From Fig. 4)

$P = A_e / Rt = 0.25$ $\theta = 48.0\ °$ (From Fig. 8)

$$T_u = \frac{310 \times 0.79\ (2.84 + 0.25 \times 4.4)\ 80.5^3 \times 4.0\ /10^3}{10^3 \times 1.00 \times 991} = \mathbf{2.02}\ \text{kNm}$$

c) Shear capacity (Refer Cl.8.2.1 , CoP Steel 2011)

$V_c = f_y\ (0.6\, A)\ / 3^{0.5}$

$= 310.0 \times 16.1\ / 3^{0.5}\ / 10 = \mathbf{288}\ \text{kN}$

d) Compressive capacity

$P_c = p_c\ A_g$ (Refer Cl. 8.7.5, CoP Steel 2011)

$p_c = p_E p_y / [f_c + (f^2_c - p_E p_y)^{0.5}]$ $(p_y = f_y)$

$l = L_E / r = 2 \times 310.0 / 6.28 = 98.7$ (L at section "C")

$p_E = \pi^2 E / l^2 = 207.7\ N/mm^2$

$\lambda_0 = 0.2(\pi^2 E / p_y)^{0.5} = 16.2\ N/mm^2$

$\alpha = 5.5$ (Curve c)

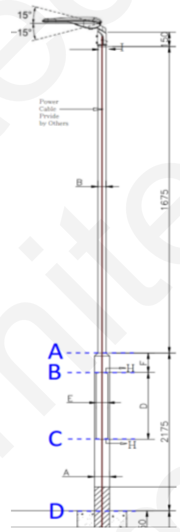
$h = \alpha (\lambda - \lambda_0)/1000 = 0.45$

$f_c = [p_y + (h+1) p_E] / 2 = 306\ N/mm^2$

$p_c = p_E p_y / [f_c + (f^2_c - p_E p_y)^{0.5}] = 135.0\ N/mm^2$

$P_c = 135.0 \times 16.1\ / 10 = \mathbf{218}\ \text{kN}$

Design for 4m Lighting Columns (Double Arm Only)

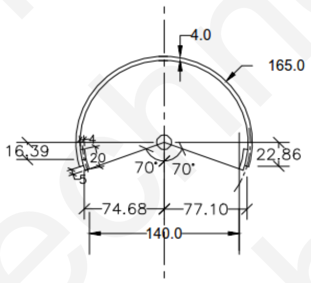


Calculation of Section Capacities

Section "B" & "C - Top & Bottom of Door Opening (with TYPE 4 Reinforcement)

Diameter	D	=	165	mm	Refer Table 7.2, CoP Steel 2011
Wall thickness	t	=	4	mm	D/t = 41.3 < 50 ε^2
Mean radius	R	=	80.5	mm	Class 2 compact
Door Opening					
Half angle of opening	θ	=	70	°	
Length	a	=	700	mm	
Corner radius	N	=	20	mm	
Effective length	L	=	691	mm	L = a - 0.43N
Half length of straight edge	C	=	660	mm	C = a - 2N

TYPE 4 door reinforcement (Refer Cl. 5.6.2.2, BS EN40-3-3:2013)



Area: 1357.8952
Perimeter: 639.3745
Moments of inertia: X: 1728806.3331
Y: 5342973.6675
Radii of gyration: X: 35.6812
Y: 62.7276

Thickness	t_w	=	4	mm
Width	d_w	=	20	mm
Area	A_e	=	80	mm

For circular sec. F = 2.0

Section Modulus $Z_{pnr} = FR^2 t\,[2\cos(\theta/2-90B_x/\pi) - \sin\theta + B_x\cos\theta] = 47.0\ cm^3$

$Z_{pyr} = FR^2 t\,(1 + \cos\theta + B_y \sin\theta) = 78\ cm^3$

$B_x = A_e / Rt * m_{ox} / m_x = 0.24$

$B_y = A_e / Rt * m_{oy} / m_y = 0.18$

Ctr of door reinf. to x axis	m_{ox}	=	74.7	mm	Col. wall edge to x	m_x	=	77.1	mm
Ctr of door reinf. to y axis	m_{oy}	=	16.4	mm	Col. wall edge to y	m_y	=	22.9	mm

Section Area	A	=	13.6	cm^2
2nd Moment of inertia	I_x	=	172.9	cm^4
	I_y	=	534.3	cm^4
Radius of gyration	r_x	=	3.57	cm
	r_y	=	6.27	cm
Design Strength	f_y	=	310	N/mm^2
Modulus of Elasticity	E	=	205	kN/mm^2
Partial Material Factor	γ_m	=	1.00	

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a) Bending Moment of Resistance

$M_{ux} = f_y\, \Phi_6\, Z_{pnr} / 10^3 \gamma_m$ (Refer Cl. 5.6.2.2, BS EN40-3-3:2013)

$M_{uy} = f_y\, \Phi_6\, Z_{pyr} / 10^3 \gamma_m$

$\Phi_6 = (2t + t_w)^2 E / [(2t + t_w)^2 E + 0.32\, RLf_y] = 0.84 \le \Phi_1 = 1.0$

$$M_{ux} = \frac{310 \times 0.8 \times 47.0}{10^3 \times 1.00} = \mathbf{12.3}\ \text{kNm}$$

$$M_{uy} = \frac{310 \times 0.8 \times 78.3}{10^3 \times 1.00} = \mathbf{20.4}\ \text{kNm}$$

b) Torsional Moment of Resistance

$T_u = f_y\, \Phi_6\, (\Phi_5 + P\, \Phi_7)\, R^3\, t / 10^3 \gamma_m L$ (Refer Cl. 5.6.2.1, BS EN40-3-3:2013)

$$\Phi_5 = \frac{10\cos^2(\theta/2)}{1+1.73\tan\theta}\left[\frac{1+2.15\tan\theta + 0.85R/L}{1+2.15\tan\theta + 0.85R/L + 3.8(R/L)^2}\right] = 1.16$$

$\Phi_7 = 4.4$ R/L = 0.116 (From Fig. 4)

$P = A_e / Rt = 0.25$ $\theta = 70.0\ ^\circ$ (From Fig. 8)

$$T_u = \frac{310 \times 0.84\ (1.16 + 0.25 \times 4.4)\ 80.5^3 \times 4.0\ /10^3}{10^3 \times 1.00 \times 691} = \mathbf{1.77}\ \text{kNm}$$

c) Shear capacity (Refer Cl.8.2.1 , CoP Steel 2011)

$V_c = f_y\ (0.6\ A)\ /3^{0.5}$

$= 310.0 \times 13.6\ /3^{0.5}\ / 10 = \mathbf{243}$ kN

d) Compressive capacity

$P_c = p_c\ A_g$ (Refer Cl. 8.7.5, CoP Steel 2011)

$p_c = p_E p_y / [f_c + (f^2_c - p_E p_y)^{0.5}]$ ($p_y = f_y$)

$l = L_E / r = 2 \times 310.0 / 6.27 = 98.8$ (L at section "C")

$p_E = \pi^2 E / l^2 = 207.1\ \text{N/mm}^2$

$\lambda_0 = 0.2(\pi^2 E/p_y)^{0.5} = 16.2\ \text{N/mm}^2$

$\alpha = 5.5$ (Curve c)

$h = \alpha(\lambda - \lambda_0)/1000 = 0.45$

$f_c = [p_y + (h+1) p_E] / 2 = 306\ \text{N/mm}^2$

$p_c = p_E p_y / [f_c + (f^2_c - p_E p_y)^{0.5}] = 134.7\ \text{N/mm}^2$

$P_c = 134.7 \times 13.6\ / 10 = \mathbf{183}$ kN

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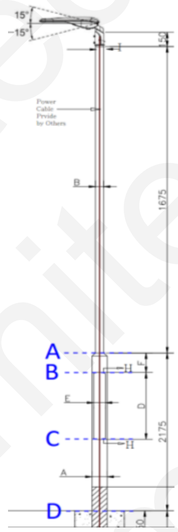
Design for 4m Lighting Columns (Single and Double Arm)

Calculation of Section Capacities



Section "D" - Base of column

Diameter	D	=	165	mm	Refer Table 7.2, CoP Steel 2011
Wall thickness	t	=	4	mm	D/t = 41.3 < 50 ε^2
Mean radius	R	=	80.5	mm	Class 2 compact
Section Area	A	=	20.2	cm^2	
Elastic Modulus	Z_p	=	$4R^2t$ = 103.7	cm^3	(Closed circular section)
2nd Moment of inertia	I	=	655.9	cm^4	
Radius of gyration	r	=	5.69	cm	
Design Strength	f_y	=	310	N/mm^2	
Modulus of Elasticity	E	=	205	kN/mm^2	
Partial Material Factor	γ_m	=	1.00		

a) Bending Moment of Resistance

$M_{ux} = M_{uy} = M_{up} = f_y \Phi_1 Z_p / 10^3 \gamma_m$ (Refer Cl. 5.6.2.1, BS EN40-3-3:2013)

$\xi = (R/t)\ \mathrm{sqrt}(f_y/E)$

$= 0.78$

$\Phi_1 = 1.0$ (Refer Figure 2 in BS EN 40-3-3:2103)

$M_{up} = \dfrac{310 \times 1.0 \times 103.7}{10^3 \times 1.00}$

$=$ **32.1** kNm

b) Torsional Moment of Resistance

$T_u = f_y \Phi_2 \pi R^2 t / 10^3 \gamma_m$ (Refer Cl. 5.6.2.1, BS EN40-3-3:2013)

$\Phi_2 = \min.\{\ 0.474\ E / f_y\ (R/t)^{1.5}\ ;\ 1.0\ \}$

$= \min.\{\ 3.47\ ;\ 1.0\ \}$

$= 1.0$

$T_u = \dfrac{310 \times 1.0\ \pi\ 80.5^2 \times 4.0\ /10^3}{10^3 \times 1.00}$

$=$ **25.2** kNm

c) Shear capacity (Refer Cl.8.2.1 , CoP Steel 2011)

$V_c = f_y\ (0.6\ A)\ /3^{0.5}$

$= 310.0 \times 12.1\ /3^{0.5}\ / 10$ = **217** kN

d) Compressive capacity

P_c	= $p_c\ A_g$			(Refer Cl. 8.7.5, CoP Steel 2011)
p_c	= $p_E p_y / [f_c + (f_c^2 - p_E p_y)^{0.5}]$			($p_y = f_y$)
l	= L_E / r = 2 x 400.0 / 5.69	=	140.5	
p_E	= $\pi^2 E / l^2$	=	102.5	N/mm^2
λ_0	= $0.2(\pi^2 E/p_y)^{0.5}$	=	16.2	N/mm^2
α	= 2.0			(Curve a)
h	= $\alpha(\lambda-\lambda_0)/1000$	=	0.25	
f_c	= $[p_y+(h+1)p_E] / 2$	=	219	N/mm^2
p_c	= $p_E p_y / [f_c + (f_c^2 - p_E p_y)^{0.5}]$	=	91.8	N/mm^2
P_c	= 91.8 x 20.2 / 10	=	**186**	kN

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Design for 4m Lighting Columns (Single and Double Arm)

Capacity Check for Single Arm Lighting Column

Case	Section	Design Forces = 1.4 * Summary table					Capacities				
		Axial	Shear	Moment	Moment	Torsion	Axial	Shear	Moment	Moment	Torsion
		N (kN)	F (kN)	M_X (kNm)	M_Y (kNm)	T (kNm)	P_c (kN)	V_c (kN)	M_{ux} (kNm)	M_{uy} (kNm)	T_u (kNm)
1.4(DL+WLX)	A	0.35	1.58	0.07	2.32	0.29	80.2	97.2	6.4	6.4	5.0
	B	0.41	1.87	0.07	2.79	0.29	217.5	288.5	16.9	23.2	2.0
	C	0.62	2.94	0.07	5.20	0.29	217.5	288.5	16.9	23.2	2.0
	D	0.82	3.89	0.07	8.27	0.29	185.7	217.3	32.1	32.1	25.2
1.4(DL+WLY)	A	0.55	1.21	1.58	0.00	0.00	80.2	97.2	6.4	6.4	5.0
	B	0.60	1.50	1.95	0.00	0.25	211.3	274.3	15.6	22.2	2.4
	C	0.75	2.57	3.99	0.00	0.39	211.3	274.3	15.6	22.2	2.4
	D	1.03	3.53	6.73	0.00	0.00	185.7	217.3	32.1	32.1	25.2

CHECK		1	2	3	4	5	Combined		Combined		
		N / Pc	F / Vc	Mx / Mux	My / Muy	T/ Tp	1 + 3 +4		1+3+4+5		
1.4(DL+WLX)	A	0.004	0.016	0.011	0.360	0.057	0.38	< 1.0	0.43	< 1.0	Okay
	B	0.002	0.006	0.004	0.120	0.142	0.13	< 1.0	0.27	< 1.0	Okay
	C	0.003	0.010	0.004	0.224	0.142	0.23	< 1.0	0.37	< 1.0	Okay
	D	0.004	0.018	0.002	0.257	0.011	0.26	< 1.0	0.28	< 1.0	Okay
1.4(DL+WLY)	A	0.007	0.012	0.246	0.000	0.000	0.25	< 1.0	0.25	< 1.0	Okay
	B	0.003	0.005	0.125	0.000	0.101	0.13	< 1.0	0.23	< 1.0	Okay
	C	0.004	0.009	0.255	0.000	0.158	0.26	< 1.0	0.42	< 1.0	Okay
	D	0.006	0.016	0.209	0.000	0.000	0.21	< 1.0	0.21	< 1.0	Okay

Capacity Check for Double Arm Lighting Column

Case	Section	Design Forces = 1.4 * Summary table					Capacities				
		Axial	Shear	Moment	Moment	Torsion	Axial	Shear	Moment	Moment	Torsion
		N (kN)	F (kN)	M_X (kNm)	M_Y (kNm)	T (kNm)	P_c (kN)	V_c (kN)	M_{ux} (kNm)	M_{uy} (kNm)	T_u (kNm)
1.4(DL+WLX)	A	0.55	2.54	0.15	4.13	0.56	80.2	97.2	6.4	6.4	5.0
	B	0.60	2.75	0.15	4.67	0.56	182.9	243.0	12.3	20.4	1.8
	C	0.75	3.50	0.15	6.86	0.56	182.9	243.0	12.3	20.4	1.8
	D	1.03	4.85	0.15	12.16	0.56	185.7	217.3	32.1	32.1	25.2
1.4(DL+WLY)	A	0.55	1.21	1.62	0.00	0.00	80.2	97.2	6.4	6.4	5.0
	B	0.60	1.43	1.89	0.00	0.32	182.9	243.0	12.3	20.4	1.8
	C	0.75	2.17	3.15	0.00	0.40	182.9	243.0	12.3	20.4	1.8
	D	1.03	3.53	6.77	0.00	0.00	185.7	217.3	32.1	32.1	25.2

CHECK		1	2	3	4	5	Combined		Combined		
		N / Pc	F / Vc	Mx / Mux	My / Muy	T/ Tp	1 + 3 +4		1+3+4+5		
1.4(DL+WLX)	A	0.007	0.026	0.023	0.643	0.110	0.67	< 1.0	0.78	< 1.0	Okay
	B	0.003	0.011	0.012	0.229	0.313	0.24	< 1.0	0.56	< 1.0	Okay
	C	0.004	0.014	0.012	0.336	0.313	0.35	< 1.0	0.67	< 1.0	Okay
	D	0.006	0.022	0.005	0.378	0.022	0.39	< 1.0	0.41	< 1.0	Okay
1.4(DL+WLY)	A	0.007	0.012	0.251	0.000	0.000	0.26	< 1.0	0.26	< 1.0	Okay
	B	0.003	0.006	0.154	0.000	0.178	0.16	< 1.0	0.34	< 1.0	Okay
	C	0.004	0.009	0.256	0.000	0.226	0.26	< 1.0	0.49	< 1.0	Okay
	D	0.006	0.016	0.211	0.000	0.000	0.22	< 1.0	0.22	< 1.0	Okay

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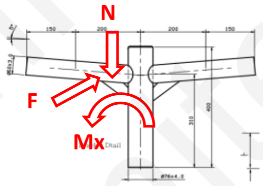
Design for 4m Lighting Columns (Single and Double Arm)



Check bracket for Lantern 60 x 3 mm thk CHS

Refer column section design,

Weight of lantern = 0.12 kN

Weight of bracket = 0.04 x 0.35 = 0.39 kN

Vertical force N = 0.12 + 0.39 = 0.51 kN

Moment M_x = 0.12 x (0.35 + 0.76 /2) + 0.51 x (0.35 /2) = 0.18 kNm

Wind x on lantern = 1.00 x 3.84 x 0.13 = 0.49 kN

Wind x on bracket = 1.20 x 3.84 x 0.06 x 0.35 = 0.10 kN

Horiz. Shear F = 0.49 + 0.10 = 0.59 kN

Moment M_y = 0.49 x (0.35 + 0.76 /2) + 0.10 x (0.35 /2) = 0.37 kNm

Wind y on lantern = 1.00 x 3.84 x 0.06 = 0.23 kN

Axial force N = 0.23 kN

Moment M_x = 0.23 x 0.233 = 0.05 kNm

Consider 1.4 DL +1.4 WLx + 1.4 Wly for conservative check

Axial N = 1.40 x 0.51 + 1.40 x 0.00 + 1.40 x 0.23 = 1.03 kN

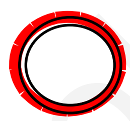
Shear F = 1.40 x 0.00 + 1.40 x 0.59 + 1.40 x 0.00 = 0.82 kN

Moment M = 1.40 x 0.18 + 1.40 x 0.37 + 1.40 x 0.05 = 0.85 kNm

Capacity Check

Refer next page

Check Weld Connection between bracket and shaft



Axial N = 1.03 kN

Shear F = 0.82 kN

Moment M = 0.85 kNm

Weld size t_w = 3.00 mm F.W. ALL ROUND

Throat size a_w = 2.10 mm

Weld radius r_w = 31.1 mm

Shear stress f_s = $\frac{(N + F)}{2\pi r_w \times a_w}$ = $\frac{1.85 \times 10^3}{409.7}$ = 4.5 N/mm^2

< 220.0 N/mm^2 **Okay**

Bending stress f_s = $\frac{M}{\pi r_w^2 \times a_w}$ = $\frac{0.85 \times 10^6}{6360.5}$ = 133.1 N/mm^2

< 220.0 N/mm^2 **Okay**

Combined $(3f_s^2 + f_b^2)^{0.5}$ = 133.4 N/mm^2 < 220.0 N/mm^2 **Okay**

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Location	4m Double Arm

Design Code	CoP for the Structural Use of Steel 2011

ULS Design Loa

Axial Compression, F		1.03	kN
Shear, V		0.82	kN
Major axis moment, M_x		0.85	kNm
Minor axis moment, M_y		0	kNm
Moment amp. (P-D)		1.00	
Major axis :	L_x	0.35	m
	L_{Ex} = 3.0 L_x	1.05	m
Minor axis :	L_y	0.35	m
	L_{Ey} = 3.0 L_y	1.05	m
LTB :	L_E = 2.0 L_x	0.7	m

Section Properties

ϕ 60x3 CHS Q345 Hot-Rolled	
Elastic critical load factor, l_{cr}	N/A
Non-sway frame	

p_y = 215 N/mm^2

E = 205 kN/mm^2

A =	5.372 cm^2	r =	2.018 cm	I =	22 cm^4
D =	60 mm	Z =	7 cm^3	S =	10 cm^3
t =	3 mm	J =	44 cm^4	C =	15 cm^3
D/t =	20				

Section Classification (Table 7.2)

e = 1.13

D/t= 20.0 < 40 e *Class 1 Plastic Section*

Shear Capacity (Cl. 8.2.1)

$V_c = p_y A_v / \sqrt{3}$ where A_v = 0.6A

= 40 kN > 0.82 kN **O.K.**

$0.6V_c$ = 24 kN > 0.82 kN **Low Shear**

Moment Capacity (Cl. 8.2.2)

For Class 1 plastic section in low shear condition,

M_{cx} = 2 kNm (Eq. 8.2)

M_{cy} = 2 kNm (Eq. 8.2)

Buckling Resistance Moment (Cl. 8.3.5)

Check on LTB is not required for CHS

Compression Resistance (Cl. 8.7.5)

For hot-rolled CHS with thickness ≤ 40mm,

use strut curves a (x-axis) & a (y-axis)

P_{cx} = 106 kN $\overline{P}_{cx}$ = 116 kN

P_{cy} = 106 kN $\overline{P}_{cy}$ = 116 kN

Cross-Section Capacity (Cl. 8.9.1)

$$\frac{F}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} = \frac{1}{116} + \frac{1}{2} + \frac{0}{2} = 0.46 < 1.0$$ **O.K.** (Eq. 8.78)

Member Buckling Resistance (Cl. 8.9.2)

$\overline{M}_x$ = 0.847 kNm P-d(x) = 1.00 M_x = 1 kNm m_x = 1.00

$\overline{M}_y$ = 0 kNm P-d(y) = 1.00 M_y = 0 kNm m_y = 1.00

$$\frac{F_c}{P_c} + \frac{m_x \overline{M}_x}{M_{cx}} + \frac{m_y \overline{M}_y}{M_{cy}} = \frac{1}{106} + \frac{1}{2} + \frac{0}{2} = 0.46 < 1.0$$ **O.K.** (Eq. 8.79)

$$\frac{F_c}{\overline{P}_c} + \frac{m_x M_x}{M_{cx}} + \frac{m_y M_y}{M_{cy}} = \frac{1}{116} + \frac{1}{2} + \frac{0}{2} = 0.46 < 1.0$$ **O.K.** (Eq. 8.80)

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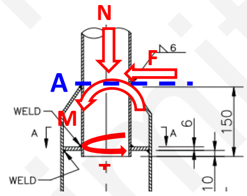
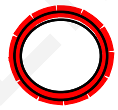
Design for 4m Lighting Columns (Single and Double Arm)

Check Weld Connection between shaft and base

Refer design forces table, max. forces at Section "A"



Axial	N	=	0.55	kN
Shear	F	=	2.54	kN
Moment	$M_x + M_y$	=	4.28	kNm
Torsion	T	=	0.56	kNm



Weld size	t_w	=	4.00	mm F.W. ALL ROUND
Throat size	a_w	=	2.80	mm
Weld radius	r_w	=	39.4	mm
Insert length	L	=	150	mm

=> Shear force due to moment

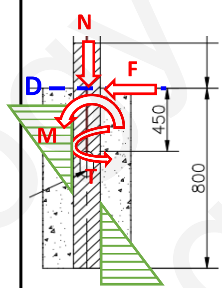
$$Fs = M / L = 4.28 / 0.15 = 28.5 \text{ kN}$$

Shear stress $f_s = \dfrac{(N + F + Fs)}{2\pi r_w \times a_w} + \dfrac{T}{2a_w \times \pi r_w^2} = \dfrac{31.63 \times 10^3}{693.2} + \dfrac{0.56 \times 10^6}{27310.5}$

$= 66.0 \text{ N/mm}^2 < 200.0 \text{ N/mm}^2$ **Okay**

Check Base

Refer design forces table, max. forces at Section "D"



Axial	N	=	1.03	kN
Shear	F	=	4.85	kN
Moment	$M_x + M_y$	=	12.31	kNm
Torsion	T	=	0.56	kNm
Radius	R_B	=	82.5	mm
Embedment	L_B	=	800	mm

Shear stress $fs = \dfrac{N}{2\pi R_B \times L_B} + \dfrac{T}{2L_B \times \pi R_B^2} = \dfrac{1.03 \times 10^3}{414690.2} + \dfrac{0.56 \times 10^6}{34211944.0}$

$= 0.02 \text{ N/mm}^2 < 0.3 \text{ N/mm}^2 \; (v_c)$ **Okay**

Bearing stress $fc = \dfrac{F}{2R_B \times L_B} + \dfrac{M + F^*L_B/2}{2R_B \times L_B^2/6} = \dfrac{4.85 \times 10^3}{132000.0} + \dfrac{14.25 \times 10^6}{17600000.0}$

$= 0.85 \text{ N/mm}^2 < 12.0 \text{ N/mm}^2 \; (0.4f_{cu})$ **Okay**

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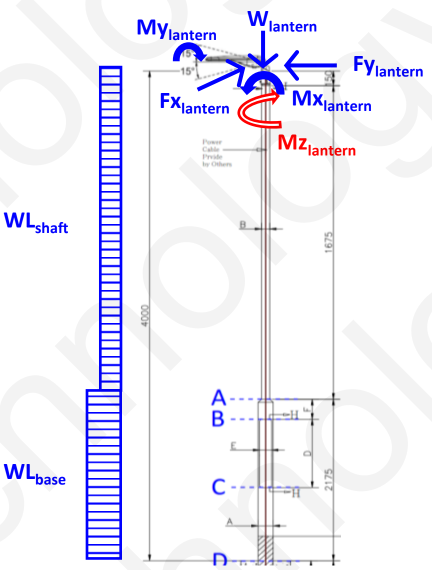
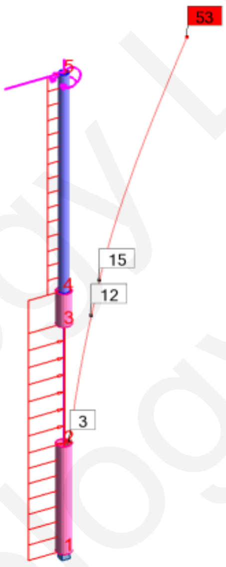
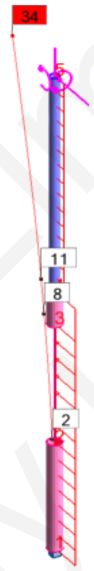
Design for 4m Lighting Columns (Single and Double Arm)

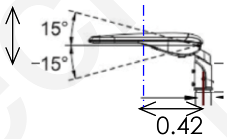
Deflection Check for Single Arm







h'= 0.364



Weight of lantern

$W_{lantern}$	=	0.12	kN					
$Mx_{Lantern}$	=	0.12	x	0.420			=	0.05 kNm

Wind Load X

WL_{shaft}	=	0.75	x	3.84	x	0.076	=	0.22 kN/m
WL_{base}	=	0.75	x	3.84	x	0.165	=	0.48 kN/m
$Fx_{Lantern}$	=	1.00	x	3.84	x	0.13	=	0.49 kN
$My_{Lantern}$	=	0.49	x	0.364			=	0.18 kNm
$Mz_{Lantern}$	=	0.49	x	0.420			=	0.21 kNm

Wind Load Y

WL_{shaft}	=	0.75	x	3.84	x	0.076	=	0.22 kN/m
WL_{base}	=	0.75	x	3.84	x	0.165	=	0.48 kN/m
$Fy_{Lantern}$	=	1.00	x	3.84	x	0.06	=	0.23 kN
$Mx_{Lantern}$	=	0.23	x	0.364			=	0.08 kNm

Refer Analysis Output

Deflection for Dead and Wind x Load
= 53 mm

Deflection for Dead and Wind y Load
= 34 mm

Allowable deflection = 0.04 (height of column + length of bracket)
= 0.04 (4.00 + 0.00)
= 160 mm > max. deflection

Okay

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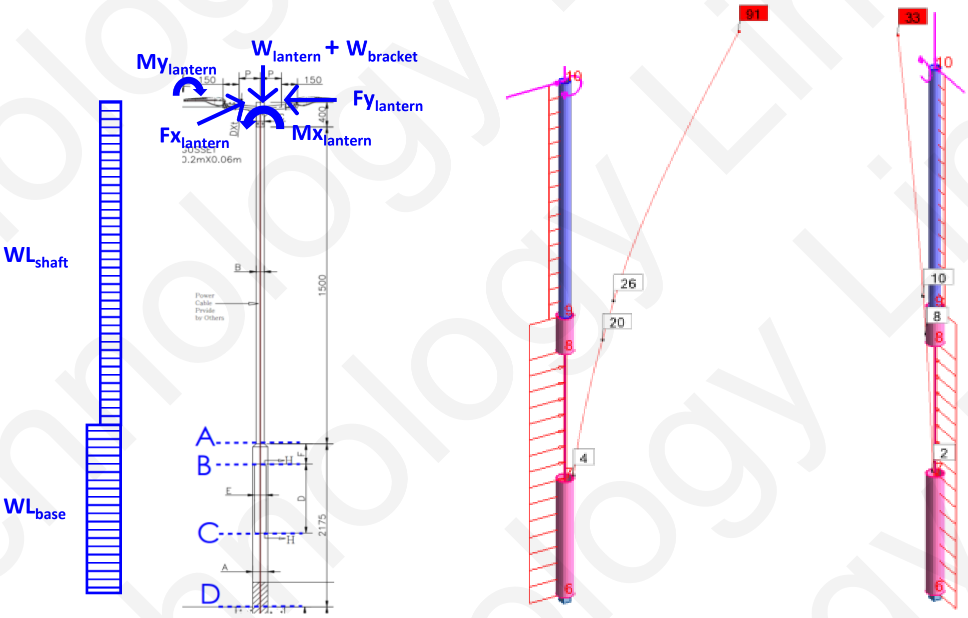
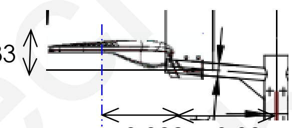
Design for 4m Lighting Columns (Single and Double Arm)

Deflection Check for Double Arm



h'= 0.233



Weight of lantern + bracket

$W_{lantern}$ = 0.12 x 2 + 0.01 x 2 = 0.27 kN

Wind Load X

WL_{shaft}	=	0.75	x	3.84	x	0.076	=	0.22 kN/m
WL_{base}	=	0.75	x	3.84	x	0.165	=	0.48 kN/m
$Fx_{Lantern}$	=	1.00	x	3.84	x	0.13 x 2	=	0.98 kN
$Fx_{bracket}$	=	0.75	x	3.84	x	0.06 x 0.35 x 2	=	0.12 kN
$My_{Lantern}$	=	0.98	x	0.233			=	0.23 kNm

Wind Load Y

WL_{shaft}	=	0.75	x	3.84	x	0.076	=	0.22 kN/m
WL_{base}	=	0.75	x	3.84	x	0.165	=	0.48 kN/m
$Fy_{Lantern}$	=	1.00	x	3.84	x	0.06	=	0.23 kN
$Mx_{Lantern}$	=	0.23	x	0.233			=	0.05 kNm

No torsion induced for lantern on both sides

Refer Analysis Output

Deflection for Dead and Wind x Load = 91 mm

Deflection for Dead and Wind y Load = 33 mm

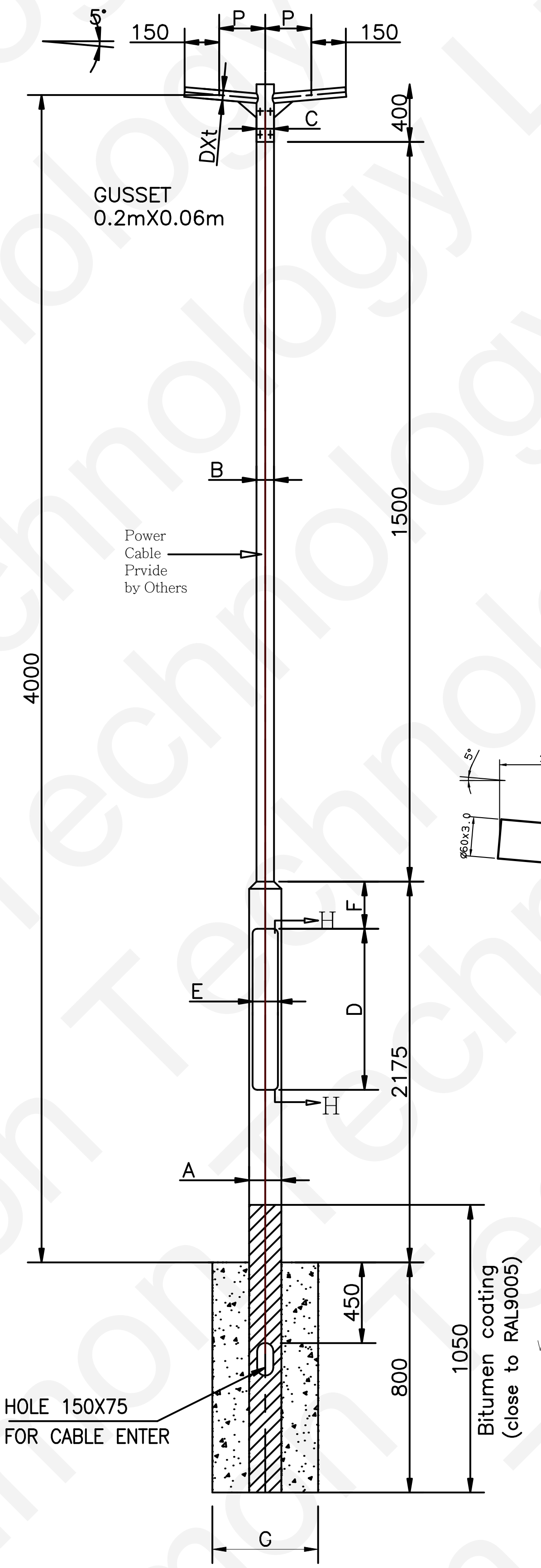
Allowable deflection = 0.04 (height of column + length of bracket)
= 0.04 (4.00 + 0.35)
= 174 mm > max. deflection

Okay

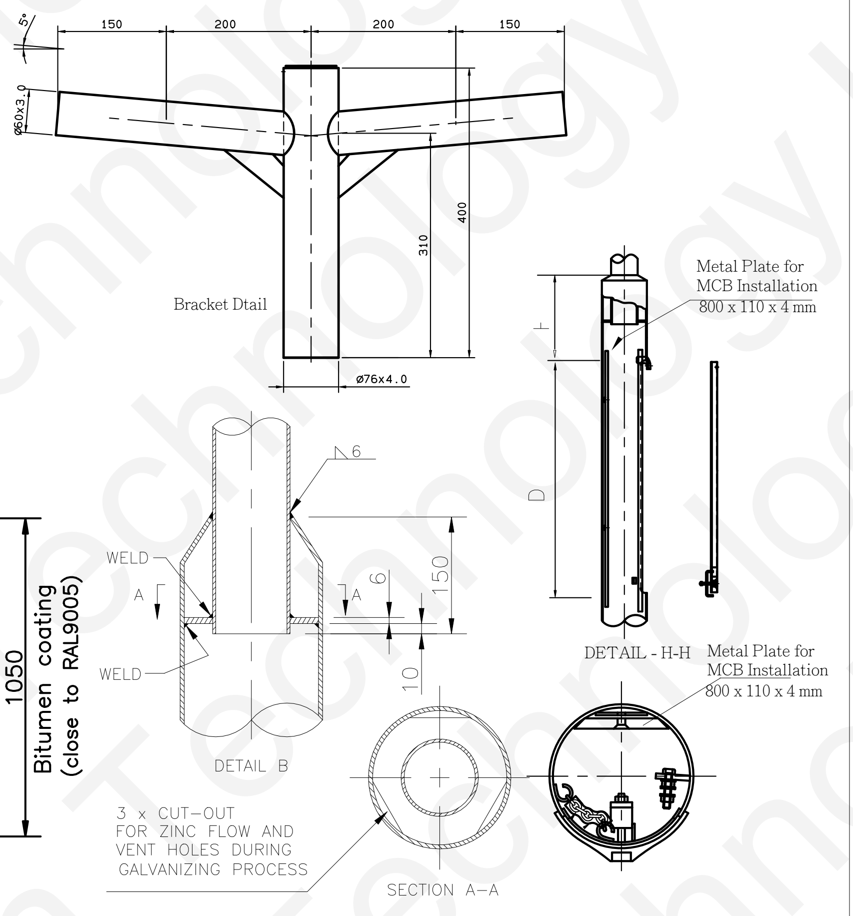
Appendix A – Drawings

4.0M 80m/s Rooted Column Twin Arm Type

*Assume lantern = 15 kg
*Area = 0.13 sqm max. for all poles



BRACKET OPTION	
BRACKET REF	BKT-051/D
P	0.2m
Dxt	60.0x3.0
C POT DIAMETER	⌀76X4.0mm THICK
COLUMN DIMENSIONS	
A BASE DIAMETER	⌀165X4.0mm THICK
B SHAFT DIAMETER	⌀76X4.0mm THICK
D DOOR HEIGHT	700mm
E DOOR WIDTH	110mm
F	205mm
CONCRETE DIMENSIONS	
G	439mm (GROUND G=230kN/sqm)
	259mm (GROUND G=390kN/sqm)
	No concrete surround reuited (GROUNG G=630kg/sqm)
FINISH	
HOT DIP GALVANISED TO ISO1461.	
Option : Coating RAL 9006 or Others	
MATERIAL	
STEEL GRADE Q345C FOR BRACKET	
STEEL GRADE Q345C FOR COLUMN	
STEEL GRADE Q345C FOR DOOR REINFORCING BAR	
STEEL GRADE Q345C FOR OTHER PLATES	
Q345C-GB/T 1591-2008	
DESIGN	
COLUMN DESIGN TO BS EN40	



Design Spec:
1. Code of Practice on Wind Effects in Hong Kong 2004 (CoP WE 2004)
2. Explanatory Materials to the Code of Practice on Wind Effects 2004 (EM CoP WE 2004)
3.Code of Practice for the Structural Use of Steel 2011 (CoP SUOS 2011)
4.BS EN 1991-1.4-2005 (2010) Wind Actions

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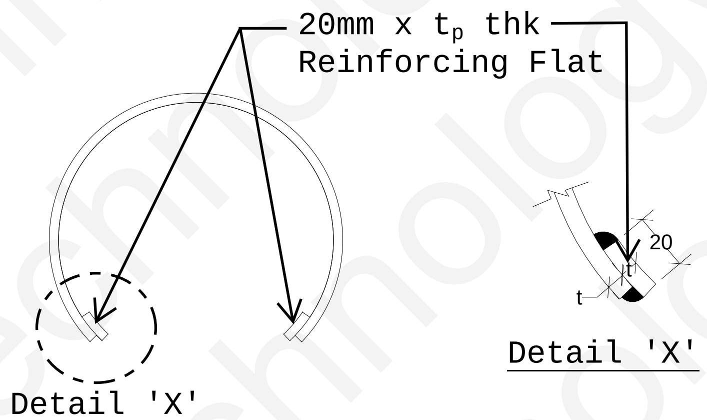
sales@dimontechnology.com

DIMON TECHNOLOGY

CLIENT				
TITLE	4.0M 80m/s Rooted Column-Twin Arm 0.2M			
DRAWN	2019-6-15	MATERIAL		MANUFACTURING ORDER
ENGR	2019-6-15	THK(mm)		
CHECKED	2019-6-15	WT(kg)		P/N HK-RC-4M-80M/S-DA
SPECIFICATIONS		SCALE		



Door Reinforcing Detail

Remarks: t_p shall be the thickness of column

SKETCH NO. SK01