STEEL COLUMN AND BASE PLATE DESIGN (EN1993)

Tedds calculation version 1.0.02

STEEL COLUMN DESIGN

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

Tedds calculation version 1.1.04

Design summary

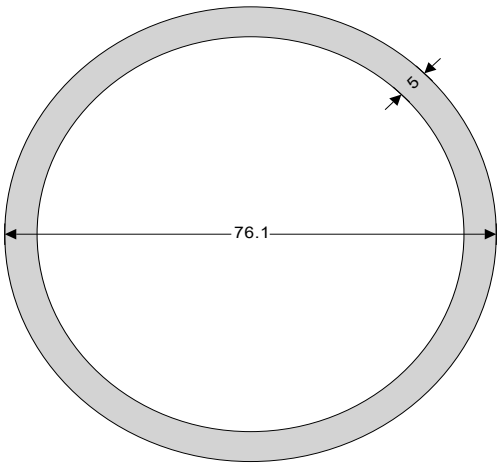
Description	Unit	Provided	Required	Utilisation	Result
Shear resistance (y-y)	kN	113	50	0.443	PASS
Axial compression	kN	307	100	0.326	PASS
Buckling in compression	kN	184	100	0.544	PASS

Partial factors - Section 6.1

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

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

Resistance of cross-sections in tension to fracture γ_{M2} = **1.1**



CHS 76.1x5.0 (Tata Steel Celsius)
Diameter, d, 76.1 mm
Mass of section, Mass, 8.8 kg/m
Section thickness, t, 5 mm
Area of section, A, 1117 mm^2
Radius of gyration about y-axis, i_y, 25.2 mm
Radius of gyration about z-axis, i_z, 25.2 mm
Elastic section modulus about y-axis, $W_{el.y}$, 18639 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 18639 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 25318 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 25318 mm^3
Second moment of area about y-axis, I_y, 709220 mm^4
Second moment of area about z-axis, I_z, 709220 mm^4

Column details

Column section **CHS 76.1x5.0**

Steel grade **S275**

Yield strength f_y = **275** N/mm^2

Ultimate strength f_u = **410** N/mm^2

Modulus of elasticity E = **210** kN/mm^2

Poisson's ratio ν = **0.3**

Shear modulus G = E / [2 × (1 + ν)] = **80.8** kN/mm^2

Column geometry

System length for buckling - Major axis L_y = **2000** mm

System length for buckling - Minor axis L_z = **2000** mm

The column is not part of a sway frame in the direction of the minor axis

The column is not part of a sway frame in the direction of the major axis

Column loading

Axial load N_{Ed} = **100** kN (Compression)

Major axis moment at end 1 - Bottom $M_{y,Ed1}$ = **0.0** kNm

Major axis moment at end 2 - Top $M_{y,Ed2}$ = **0.0** kNm

Minor axis moment at end 1 - Bottom $M_{z,Ed1}$ = **0.0** kNm

Minor axis moment at end 2 - Top $M_{z,Ed2}$ = **0.0** kNm

Major axis shear force $V_{y,Ed}$ = **50** kN

Minor axis shear force $V_{z,Ed}$ = **0** kN

Buckling length for flexural buckling - Major axis

End restraint factor K_y = **1.200**

Buckling length $L_{cr\_y} = L_y \times K_y$ = **2400** mm

Buckling length for flexural buckling - Minor axis

End restraint factor K_z = **1.200**

Buckling length $L_{cr\_z} = L_z \times K_z$ = **2400** mm

Section classification (Table 5.2)

Coefficient depending on f_y $\varepsilon = \sqrt{(235\ N/mm^2 / f_y)}$ = **0.924**

Ratio of d/t ratio = d / t = **15.22**

Limit of d/t for class 1 section $Limit_1 = 50 \times \varepsilon^2$ = **42.73**

Limit of d/t for class 2 section $Limit_2 = 70 \times \varepsilon^2$ = **59.82**

Limit of d/t for class 3 section $Limit_3 = 90 \times \varepsilon^2$ = **76.91**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear - Major axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = **50.0** kN

Shear area $A_{vy} = 2 \times A / \pi$ = **711** mm^2

Plastic shear resistance $V_{pl,y,Rd} = A_{vy} \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **112.9** kN

$V_{y,Ed} / V_{pl,y,Rd}$ = **0.443**

PASS - Shear resistance exceeds the design shear force

$V_{y,Ed}$ <= 0.5×$V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = **100** kN

Design resistance $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **307** kN

$N_{Ed} / N_{c,Rd}$ = **0.326**

PASS - The compression design resistance exceeds the design force

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance f_y = **275** N/mm^2

Flexural buckling - Major axis

Elastic critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr\_y}^2$ = **255** kN

Non-dimensional slenderness $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **1.097**

Buckling curve (Table 6.2) **a**

Imperfection factor (Table 6.1) α_y = **0.21**

Parameter Φ $\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$ = **1.196**

Reduction factor $\chi_y = \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}])$ = **0.598**

Design buckling resistance $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **183.7** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.544**

PASS - The flexural buckling resistance exceeds the design axial load

Flexural buckling - Minor axis

Elastic critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr\_z}^2$ = **255** kN
Non-dimensional slenderness	$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **1.097**
Buckling curve (Table 6.2)	**a**
Imperfection factor (Table 6.1)	α_z = **0.21**
Parameter Φ	$\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$ = **1.196**
Reduction factor	$\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}])$ = **0.598**
Design buckling resistance	$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **183.7** kN
	$N_{Ed} / N_{b,z,Rd}$ = **0.544**

PASS - The flexural buckling resistance exceeds the design axial load

Minimum buckling resistance

Minimum buckling resistance	$N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd})$ = **183.7** kN
	$N_{Ed} / N_{b,Rd}$ = **0.544**

PASS - The axial load buckling resistance exceeds the design axial load

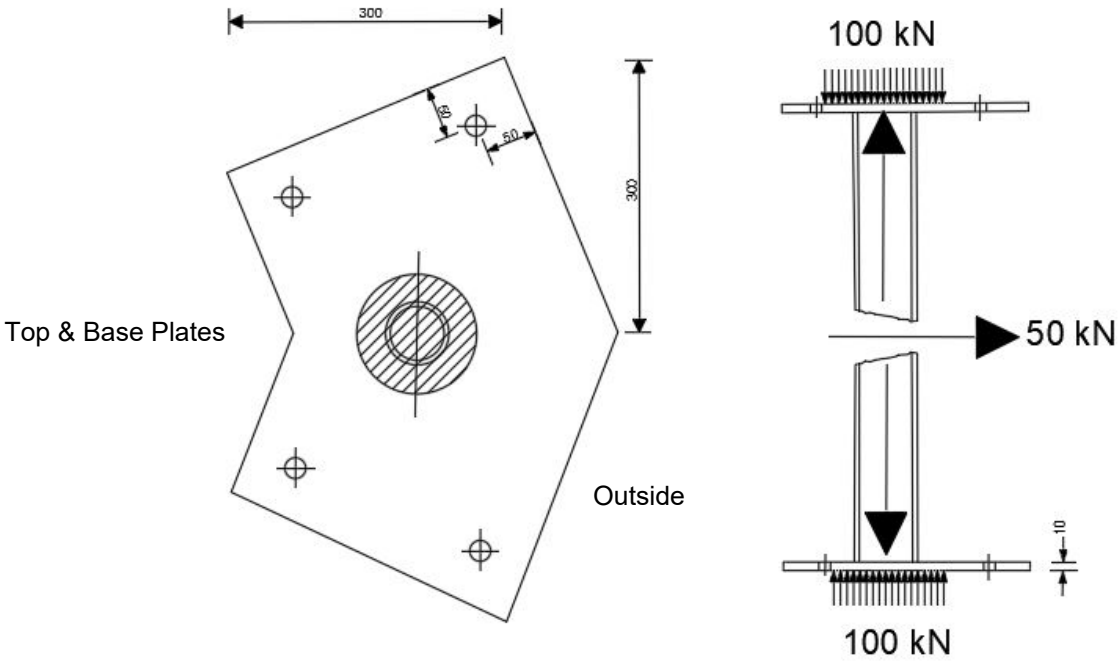
COLUMN TOP & BASE PLATE DESIGN

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009, and EN1993-1-8:2005 incorporating Corrigenda December 2005, September 2006 and July 2009, and the UK national annex

Tedds calculation version 2.0.06

Design summary

Description	Unit	Design	Resistance	Utilisation	Result
Axial	kN	100	126.1	0.79	PASS
Weld shear	kN	50	226	0.22	PASS



Design forces

Design axial force (compression)	N_{Ed} = **100** kN
Design shear force	V_{Ed} = **50** kN
Design moment	M_{Ed} = **0** kNm

Column Details

Column section	**CHS 76.1x5.0**
Depth	D = **76.1** mm
Thickness	t = **5** mm

Base plate details

Length	h_p = **300** mm
Width	b_p = **600** mm
Thickness	t_p = **10** mm
Column eccentricity in x-axis	e_{bpx} = **-25** mm

Bolt/Anchor details

Number of anchors LHS	n_1 = **2**
Edge distance in x-axis	e_{x1} = **50** mm
Edge distance in y-axis	e_{y1} = **50** mm
Number of anchors RHS	n_2 = **2**
Edge distance in x-axis	e_{x2} = **50** mm
Edge distance in the y-axis	e_{y2} = **50** mm
Anchor diameter	$d_{a,b}$ = **12** mm M12 Grade 8.8

Foundation details

Bottom edge distance	c_1 = **600** mm
Right edge distance	c_2 = **100** mm
Left edge distance	c_3 = **100** mm
Top edge distance	c_4 = **600** mm
Foundation depth	t_{fnd} = **500** mm

Concrete details

Concrete strength class	**C12/15**
Characteristic compressive cylinder strength	f_{ck} = **12** N/mm^2
Characteristic compressive cube strength	$f_{ck,cube}$ = **15** N/mm^2
Partial factor for concrete	γ_c = **1.50**
Compressive strength coefficient	α_{cc} = **0.85**
Design compressive concrete strength	$f_{cd} = \alpha_{cc} \times (f_{ck} / \gamma_c)$ = **6.8** N/mm^2

Steel details

Base plate steel grade	**S275**
Base plate nominal yield strength	f_{yp_plt} = **275** N/mm^2
Base plate nominal ultimate tensile strength	f_{u_plt} = **410** N/mm^2
Column steel grade	**S275**
Column nominal yield strength	f_{yp_col} = **275** N/mm^2
Column nominal ultimate tensile strength	f_{u_col} = **410** N/mm^2
Partial safety factor cross sections	γ_{M0} = **1.00**
Partial safety factor welds	γ_{M2} = **1.25**

Foundation bearing strength - EN1992-1-1 Section 6.7

Loaded area	$A_{c0} = b_p \times h_p$ = **150000** mm^2

Design distribution width	$b_{p,dist}$ = **833.3** mm
Design distribution length	$h_{p,dist}$ = **500** mm
Maximum design distribution area	$A_{c1} = b_{p,dist} \times h_{p,dist}$ = **416667** mm^2
Concentrated design resistance force	$F_{Rdu} = Min(A_{c0} \times f_{cd} \times \sqrt{(A_{c1} / A_{c0})}, 3 \times f_{cd} \times A_{c0})$ = **1700** kN
Foundation joint material coefficient	β_j = **0.67**
Design bearing strength of the joint	$f_{jd} = \beta_j \times F_{Rdu} / (b_p \times h_p)$ = **7.56** N/mm^2

Base plate compressive resistance

Additional bearing width	$c = t_p \times \sqrt{(f_{yp\_plt} / (3 \times f_{jd} \times \gamma_{M0}))}$ = **34.8** mm
Effective bearing area	A_{eff} = **16687** mm^2
Design compressive resistance	$N_{c,Rd} = f_{jd} \times A_{eff}$ = **126.1** kN
	PASS - Design compressive resistance exceeds applied axial load

Shear weld

Force in shear weld	$F_{w,v,Ed}$ = **50** kN
Weld leg length	s_w = **6** mm
Weld throat size	$a_w = 1 / \sqrt{(2)} \times s_w$ = **4.2** mm
Length of weld	$L_{w,v} = \pi \times D$ = **239.1** mm
Correlation factor for fillet welds - Table 4.1	β_w = **0.85**
Design shear strength - Cls 4.5.3.3(3)	$f_{vw,d} = f_{u\_plt} / (\sqrt{(3)} \times \beta_w \times \gamma_{M2})$ = **222.8** N/mm^2
Design resistance per unit length - Cls 4.5.3.3(2)	$f_{w,Rd} = f_{vw,d} \times a_w$ = **945.2** N/mm
Design resistance	$F_{w,v,Rd} = f_{w,Rd} \times L_{w,v}$ = **226** kN
	PASS - Available strength of weld exceeds force in weld