C13

unBraced slender

General column design by *PROKON.* (GenCol Ver W4.0.06 - 08 Feb 2021)

Design code : Eurocode 2 - 2004

Input tables

General design parameters:

COLUMN SECTION		
Code	X/Radius or Bar Diameter (mm)	Y (mm) Angle °
+	25.000	
	150.000	
	25.000	25.000
		350.000
	-25.000	25.000
	-150.000	
	-25.000	-25.000
		-350.000
+	42.500	42.500
b	25	
+	157.500	42.500
b	25	
+	157.500	357.500
b	25	
+	42.500	357.500
b	25	

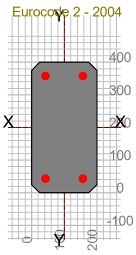
Design loads:

LOADS (ULTIMATE LIMIT STATE)						
Load case	Designation	P (kN)	Mx Top (kNm)	My Top (kNm)	Mx Bottom (kNm)	My Bottom (kNm)
1		100	20	4	10	2

Code specific parameters:

Building inclination rad	0.005
Effective creep ratio - 5.8.4	0

General design parameters:



Given:

l = 15.000 m
fck = 15 MPa
fy = 500 MPa
Ac = 78750 mm²

Assumptions:
(1) The general conditions of clause 5.8.8.2 are applicable.
(2) The specified design axial loads include the self-weight of the column.
(3) The design axial loads are taken constant over the height of the column.

Design approach:
The column is designed using an iterative procedure:
(1) The column design charts are constructed.
(2) An area of reinforcement is chosen.
(3) The corresponding slenderness moments are calculated.
(4) The design axis and design ultimate moment are determined.
(5) The design axial force and moment capacity is checked on the relevant design chart.
(6) The safety factor is calculated for this load case.

Check column slenderness:
End fixity and bracing for bending about the Design axis:
The column is unbraced.

Effective length factor ß = 1.20

Effective column height:

$$l_o = \beta \cdot l$$
$$= 1.2 \times 15$$
$$= 18.000 \text{ m}$$

Column slenderness about weakest axis:

$$m_{ax\_sl} = \frac{l_e}{r}$$
$$= \frac{18}{.05703}$$
$$= 315.623$$

Minimum Moments for Design:
Check for mininum eccentricity:
Check that the eccentricity exceeds the minimum in the plane of bending:

5.2(7)

$$e_{minx} = \frac{\theta \cdot l_0}{2}$$
$$= \frac{.005 \times 18}{2}$$
$$= 0.0450 \text{ m}$$

5.2(7)

$$e_{miny} = \frac{\theta \cdot l_0}{2}$$

$$= \frac{.005 \times 18}{2}$$

$$= 0.0450 \text{ m}$$

Use emin = 20mm *6.1 (4)*

$$M_{minx} = e_{minx} \cdot N$$

$$= .045 \times 100$$

$$= 4.500 \text{ kNm}$$

Check if the column is slender: *5.8.3.1*

$$N_{ed} = 100.0 \text{ kN}$$

$$f_{cd} = \frac{a_{cc} \cdot f_{ck}}{1.50}$$

$$= \frac{1 \times 15}{1.50}$$

$$= 10.000 \text{ MPa}$$

$$f_{yd} = \frac{f_y}{1.15}$$

$$= \frac{500}{1.15}$$

$$= 434.783 \text{ MPa}$$

5.5.3.1 (1)

$$n = \frac{N_{ed}}{A_c \cdot f_{cd}}$$

$$= \frac{100000}{78750 \times 10}$$

$$= 0.1270$$

Check slenderness limit: 5.8.3.1 (1)

$A = 0.7$

5.8.8.3 (3)

$$\omega = \frac{A_s \cdot f_{yd}}{A_c \cdot f_{cd}}$$

$$= \frac{1\,963.5 \times 434.78}{78750 \times 10}$$

$$= 1.084$$

5.8.3.1 (1)

$$B = \sqrt{1 + 2 \cdot \omega}$$

$$= \sqrt{1 + 2 \times 1.0841}$$

$$= 1.780$$

Larger end moment

$M_{02} = 20.4$ kNm

Smaller end moment

$M_{01} = 10.2$ kNm

5.8.3.1 (1)

$$r_m = \frac{M_{01}}{M_{02}}$$

$$= \frac{10.198}{20.396}$$

$$= 0.5000$$

Unbraced therefore set rm = 1

5.8.3.1 (1)

$C = 1.7 - r_m$
$= 1.7 - 1$
$= 0.7000$

5.8.3.1 (1)

$$\lambda_{lim} = \frac{20 \cdot A \cdot B \cdot C}{\sqrt{n}}$$
$$= \frac{20 \times .7 \times 1.7799 \times .7}{\sqrt{.12698}}$$
$$= 48.950$$

λ = 315.63 > 48.95

Thus:The column is slender.

Initial moments::

The column is bent in double curvature about the X-X axis:

M1 = Smaller initial end moment = -10.0 kNm

M2 = Larger initial end moment = 20.0 kNm

The initial moment near mid-height of the column : *5.8.8.2*

$M_i = 0.4 \cdot M_1 + 0.6 \cdot M_2$
$= 0.4 \times -10 + 0.6 \times 20$
$= 8.000$ kNm

$M_{i2} = 0.4 \cdot M_2$
$= 0.4 \times 20$
$= 8.000$ kNm

Thus:Mi ≥ 0.4M2 = 8.0 kNm

The column is bent in double curvature about the Y-Y axis:

M1 = Smaller initial end moment = -2.0 kNm

M2 = Larger initial end moment = 4.0 kNm

The initial moment near mid-height of the column : *5.8.8.2*

$M_i = 0.4 \cdot M_1 + 0.6 \cdot M_2$
$= 0.4 \times -2 + 0.6 \times 4$
$= 1.600$ kNm

$M_{i2} = 0.4 \cdot M_2$
$= 0.4 \times 4$
$= 1.600$ kNm

Thus:Mi ≥ 0.4M2 = 1.6 kNm

Deflection induced moments:

5.8.8.2
5.8.8.3 (3)

$n = \frac{N_{ed}}{A_c \cdot f_{cd}}$
$= \frac{100000}{78750 \times 10}$
$= 0.1270$

5.8.8.3 (3)

$\omega = \frac{A_s \cdot f_{yd}}{A_c \cdot f_{cd}}$
$= \frac{1\,963.5 \times 434.78}{78750 \times 10}$
$= 1.084$

5.8.8.3 (3)

$n_u = 1 + \omega$
$= 1 + 1.0841$
$= 2.084$

For bending about the X-X axis:

5.8.8.3 (3)

$$n_{bal} = \frac{N_{bal}}{A_c \cdot f_{cd}}$$

$$= \frac{264401}{78750 \times 10}$$

$$= 0.3357$$

5.8.8.3 (3)

$$K_r = \frac{n_u - n}{n_u - n_{bal}}$$

$$= \frac{2.0841 - .12698}{2.0841 - .33575}$$

$$= 1.119$$

K_r is limited to <= 1

Allowable tensile strain in steel 5.8.8.3 (1)

$$\varepsilon_{yd} = \frac{f_{st}}{E_s}$$

$$= \frac{434.78}{200000}$$

$$= 0.0022$$

5.8.8.3 (4)

$$\beta = \left[0.35 + \frac{f_{ck}}{200}\right] - \frac{\lambda}{150}$$

$$= \left[0.35 + \frac{15}{200}\right] - \frac{315.63}{150}$$

$$= -1.6792$$

5.8.8.3 (4)

$$K_\phi = 1 + \beta \cdot \phi_{ef}$$

$$= 1 + -1.6792 \times 0$$

$$= 1.0000$$

$K\phi$ is limited to >= 1

5.8.8.3 (1)

$$Curvature = \frac{K_r \cdot K_\phi \cdot \varepsilon_{yd}}{0.45 \cdot d}$$

$$= \frac{1 \times 1 \times .00217}{0.45 \times .1575}$$

$$= 0.0306$$

5.8.8.2 (3)

$$e_2 = \frac{Curvature \cdot l_o^2}{c}$$

$$= \frac{.03067 \times 18^2}{10}$$

$$= 0.9937$$

5.8.8.2 (3)

$$M_{add} = N_{ed} \cdot e_2$$

$$= 100 \times .99379$$

$$= 99.379 \text{ kNm}$$

Thus:Maddx = Madd*cos(-90.00°) = 0.0 kNm
Thus:Maddy = Madd*sin(-90.00°) = 99.4 kNm

Design ultimate load and moment:

Design axial load:
P_u = 100.0 kN

Moments as a result of imperfections added about the design axis *5.8.9 2)*

For bending about the X-X axis, the maximum design moment is the greatest of: *3.8.3.7*
(a) 5.8.8.2

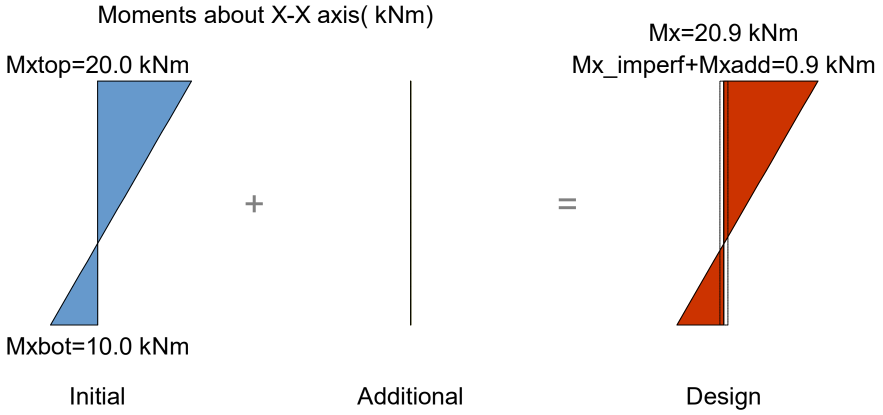
$$M = M_{top} + M_{add} + M_{imperf}$$

$$= 20 + 0 + .85474$$

$$= 20.855 \text{ kNm}$$

Thus: 5.8.8.2

$$M = 20.9 \text{ kNm}$$



Moments as a result of imperfections added about the design axis 5.8.9 2)

For bending about the Y-Y axis, the maximum design moment is the greatest of: 3.8.3.7
(a) 5.8.8.2

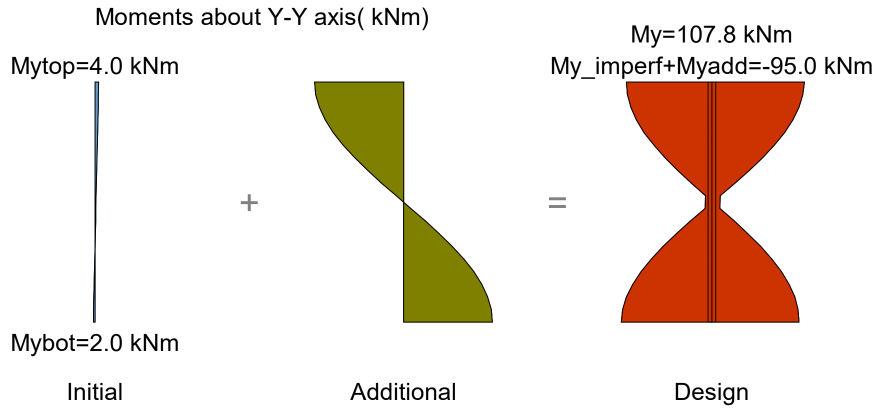
$$M = M_{top} + M_{add} + M_{imperf}$$

$$= 4 + -99.379 + 4.4181$$

$$= -90.9609 \text{ kNm}$$

Thus: 5.8.8.2

$$M = 107.8 \text{ kNm}$$



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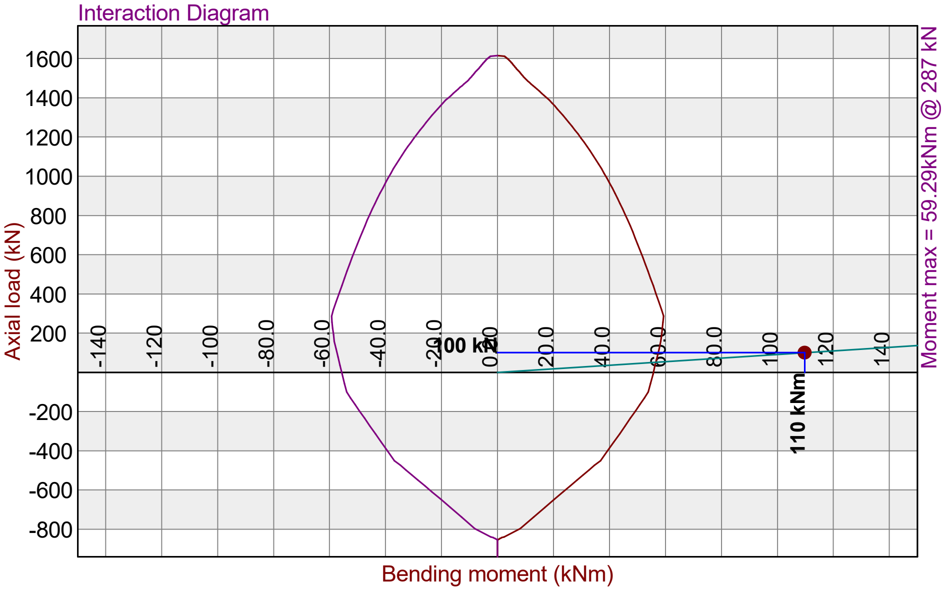
Design of column section for ULS:

The column is checked for applied moment about the design axis.
Through inspection: the critical section lies at the top end of the column.
The design axis for the critical load case 1 lies at an angle of 79.05° to the X-axis
The safety factor for the critical load case 1 is 0.54

For bending about the design axis:



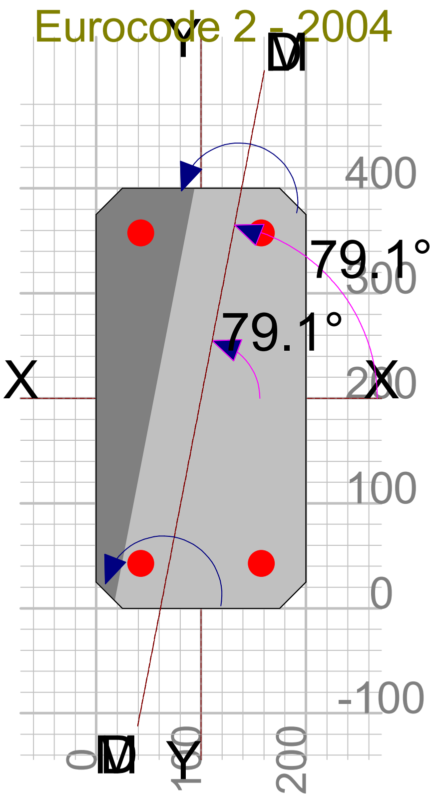
Warning: The safety factor is < 1

Moment distribution along the height of the column for bending about the design axis:

The final design moments were calculated as the vector sum of the X- and Y- moments of the critical load case. This also determined the design axis direction

At the top, Mx = 109.8 kNm
Near mid-height, Mx = 12.7 kNm
At the bottom, Mx = 106.4 kNm

Stresses at the top end of the column for the critical load case 1



Summary of design calculations:

Design table for critical load case:

Moments and Reinforcement for LC 1:				
		Top	Middle	Bottom
Madd-x	(kNm)	0.0	0.0	0.0
Madd-y	(kNm)	-99.4	0.0	99.4
Mx	(kNm)	20.9	12.4	10.4
My	(kNm)	107.8	2.5	105.9
M_imperf	(kNm)	4.5	0.0	4.5
M-design	(kNm)	109.8	12.7	106.4
Design axis (°)		79.05	11.31	264.37
Safety factor		0.54	8.74	0.56
As	(mm²)	1963	1963	1963
Percentage		2.43 %	2.43 %	2.43 %
Nominal mm^2		158	158	158
Critical load case: LC 1				

Design results for all load cases:

Load case	axis	N (kN)	M1 (kNm)	M2 (kNm)	Mi (kNm)	Madd (kNm)	Design	M (kNm)	M' (kNm)	Safety factor
Load case 1	X-X Y-Y	100.0	-10.0 -2.0	20.0 4.0	8.0 1.6	0.0 -99.4	Top	20.9 107.8	109.8	0.542