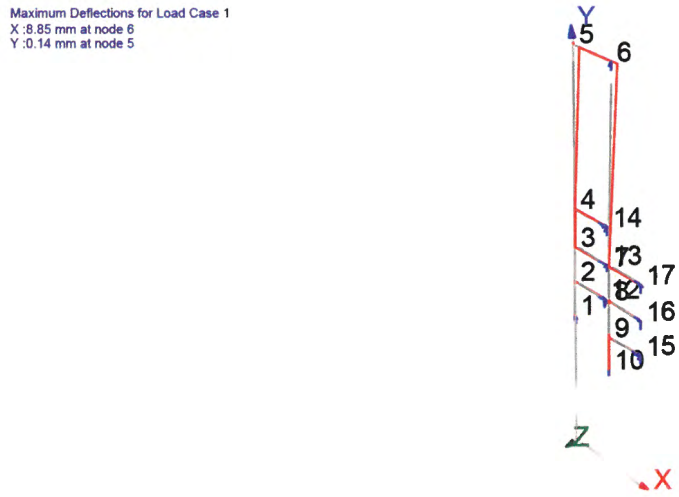
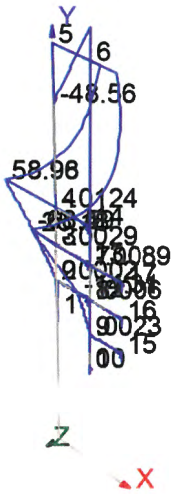
Frame Analysis Ver W3.1.24 - 17 Oct 2019
Input file: C:\delo\Prokon\Data\demo\Grušovje1.A03
Created : 11. 03. 2020 15:46:12

A03

Deflections for Load Case 1



X-Moments for Load Case 1



========== S p a c e - F r a m e A n a l y s i s - P R O K O N ==========
Ver W3.1.24 - 17 Oct 2019

TITLE :

Data file : C:\delo\Prokon\Data\demo\Grušovje1.A03
Created on: 11. 03. 2020

========== NODAL POINT COORDINATES ==========

Node No	X-coord m	Y-coord m	Z-coord m	Node No	X-coord m	Y-coord m	Z-coord m
1	0.000	4.000	0.000	2	0.000	5.000	0.000
3	0.000	6.000	0.000	4	0.000	7.000	0.000
5	0.000	11.000	0.000	6	1.100	11.000	0.000
7	1.100	6.000	0.000	8	1.100	5.000	0.000
9	1.100	4.000	0.000	10	1.100	3.000	0.000
12	1.000	5.000	0.000	13	1.000	6.000	0.000
14	1.000	7.000	0.000	15	2.100	4.000	0.000
16	2.100	5.000	0.000	17	2.100	6.000	0.000

========== ELEMENT DATA ==========

Beam	Secn. type	Fixity	Length m	ß (°)	Group no.	Stage
1-2	1	00	1.000	0.00	0	0
2-3	1	00	1.000	0.00	0	0
3-4	1	00	1.000	0.00	0	0
4-5	1	00	4.000	0.00	0	0
6-7	1	00	5.000	0.00	0	0
7-8	1	00	1.000	0.00	0	0
8-9	1	00	1.000	0.00	0	0
9-10	1	00	1.000	0.00	0	0
5-6	2	00	1.100	0.00	0	0
2-12	3	00	1.000	0.00	0	0
3-13	3	00	1.000	0.00	0	0
4-14	3	00	1.000	0.00	0	0
9-15	3	00	1.000	0.00	0	0
8-16	3	00	1.000	0.00	0	0
7-17	3	00	1.000	0.00	0	0

========== BEAM SECTION PROPERTIES ==========

Section : 1 Section designation:

A m^2	Ay m^2	Ax m^2	Ixx m^4	Iyy m^4	J m^4	Material
126.0E-3	126.0E-3	0.000	1.26E-3	0.00	0.00	Concrete:30 MPa

Section : 2 Section designation:

A m^2	Ay m^2	Ax m^2	Ixx m^4	Iyy m^4	J m^4	Material
500.0E-3	500.0E-3	0.000	62.5E-3	0.00	0.00	Concrete:30 MPa

Section : 3 Section designation:

A m^2	Ay m^2	Ax m^2	Ixx m^4	Iyy m^4	J m^4	Material
4.121E-3	4.121E-3	0.000	96.5E-9	0.00	0.00	Concrete:30 MPa

========== MATERIALS ==========

Designation	E kPa	poisson	Density kN/m^3	Exp. coeff.
Concrete:30 MPa	26.00E6	0.20	25.00	10.00E-6

========== SUPPORT DATA ==========

Prescribed displacements

Node	Fixity	X m	Y m	Z m	X-Rot rad.	Y-Rot rad.	Z-Rot rad.	Orien node
1	Y	0.00	0.00	0.00	0.00	0.00	0.00	0
10	Y	0.00	0.00	0.00	0.00	0.00	0.00	0
6	Y	0.00	0.00	0.00	0.00	0.00	0.00	0
7	X	0.00	0.00	0.00	0.00	0.00	0.00	0
12	XY	0.00	0.00	0.00	0.00	0.00	0.00	0
13	XY	0.00	0.00	0.00	0.00	0.00	0.00	0
14	XY	0.00	0.00	0.00	0.00	0.00	0.00	0
15	XY	0.00	0.00	0.00	0.00	0.00	0.00	0
16	XY	0.00	0.00	0.00	0.00	0.00	0.00	0
17	XY	0.00	0.00	0.00	0.00	0.00	0.00	0

Spring constants

Node	Fixity	X kN/m	Y kN/m	Z kN/m	X-Rot kNm/rad	Y-Rot kNm/rad	Z-Rot kNm/rad

LOADS

Load Case Description

1

Own weight not added to any load case/combination

LOAD CASE 1

*** BEAM ELEMENT LOADS ***

Element	Dir	P kN	a m	Wl kN/m	Wr kN/m	a(left) m	Length m	dT °C	Stage
4-5	X	0.00	0.00	23.5	3.50	0.00	4.00	0.00	1
6-7	X	0.00	0.00	3.50	23.5	0.00	5.00	0.00	1

OWN WEIGHT ACCELERATIONS

Direction	Acceleration (g)
X	0.00
Y	-1.00
Z	0.00

OUTPUT: LINEAR ANALYSIS

NODAL POINT DISPLACEMENTS at SLS

Node	Lcase	X-disp. mm	Y-disp mm	Z-rot. rad.
1	1	-0.28	0.00	0.0000
2	1	-0.23	0.02	0.0000
3	1	-0.08	0.04	-0.0004
4	1	0.88	0.06	-0.0017
5	1	8.85	0.14	-0.0002
6	1	8.85	0.00	-0.0001
7	1	0.00	0.00	-0.0012
8	1	-0.37	0.00	0.0001
9	1	-0.13	0.00	0.0003
10	1	0.17	0.00	0.0003
12	1	0.00	0.00	0.0000
13	1	0.00	0.00	0.0002
14	1	0.00	0.00	0.0008
15	1	0.00	0.00	-0.0002
16	1	0.00	0.00	0.0000
17	1	0.00	0.00	0.0006

REACTIONS AT ULS

(In rotated axes where applicable)

Note:Only load combinations have ULSses = 1

Node	Lcase	X-force kN	Y-forcet kN	Z-moment kNm
1	1	0.00	-67.65	0.00

10	1	0.00	0.00	0.00
6	1	0.00	67.63	0.00
7	1	-115.13	0.00	0.00
12	1	25.18	0.00	0.00
13	1	8.61	0.00	0.00
14	1	-94.33	0.01	0.00
15	1	14.31	0.00	0.00
16	1	39.86	0.00	0.00
17	1	0.00	0.01	0.00

EQUILIBRIUM CHECK AT ULS:

LC APPLIED LOADS & MOMENTS about (0.0,0.0,0.0) in global axes

Sum of:	Px	Py	Mz
1	121.50	0.00	-991.42

LC REACTIONS & REACTION MOMENTS about (0.0,0.0,0.0) in global axes

Sum of:	Rx	Ry	MRz
1	-121.50	0.00	991.42

BEAM ELEMENT END FORCES IN LOCAL ELEMENT AXES at ULS

Elem	Lcase	Axial kN	Y-Shea kN	M-xx kNm	Axial kN	Y-Shea kN	M-xx kNm
1-2	1	-67.65	0.00	0.00	67.65	0.00	0.00
2-3	1	-67.65	-25.18	0.00	67.65	25.18	-25.18
3-4	1	-67.64	-33.78	25.18	67.64	33.78	-58.96
4-5	1	-67.63	60.55	58.98	67.63	-6.55	48.56
6-7	1	0.00	-6.55	25.84	0.00	-60.95	68.50
7-8	1	-0.01	-54.18	-68.49	0.01	54.18	14.31
8-9	1	-0.01	-14.31	-14.31	0.01	14.31	0.00
9-10	1	0.00	0.00	0.00	0.00	0.00	0.00
5-6	1	-6.55	-67.63	-48.56	6.55	67.63	-25.84
2-12	1	-25.18	0.00	0.00	25.18	0.00	0.00
3-13	1	-8.61	0.00	0.00	8.61	0.00	0.00
4-14	1	94.33	-0.01	-0.01	-94.33	0.01	0.00
9-15	1	-14.31	0.00	0.00	14.31	0.00	0.00
8-16	1	-39.86	0.00	0.00	39.86	0.00	0.00
7-17	1	0.00	-0.01	-0.01	0.00	0.01	0.00

STATISTICAL DATA

Weights of beam elements: (Not included in analysis)

Section	Designation	Total Mass (kN)	(kg)
1		47.250	4816.5
2		13.750	1401.6
3		0.618	63.0
		61.618	6281.2

Total weight of structure = 0.00 kN

No. of real numbers in Stiffness matrix = 1274 (12740 bytes)
Time used to analyse = 0: 0:0.000 seconds
Total number of : Nodes = 16
Beam Elements = 15
Cable Elements = 0
Shell Elements = 0
Supports = 10
Section Properties = 3
Load Cases = 1
Load Combinations = 0

END OF OUTPUT

PROKON Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Number		Sheet
	Job Title	Grusovje6 - armatura pilotne stene	
	Client	Obcina Oplotnica	
	Calcs by	Checked by	Date 11. 03. 2020

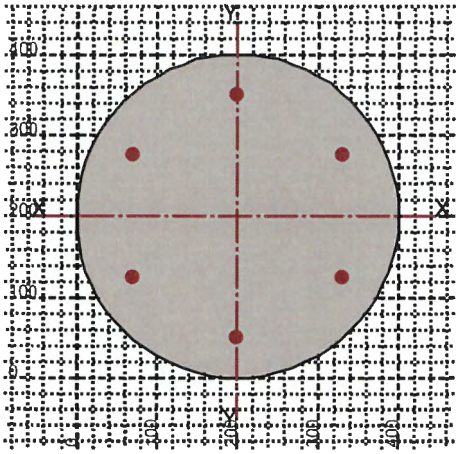
C12

Example: Cantilever column with bi-axial bending

Circular column design by *PROKON*. (CirCol Ver W3.1.06 - 17 Oct 2019)

Design code : BS8110 - 1997

General design parameters:



Given:
d = 400 mm
d' = 50 mm
Lo = 7.600 m
fcu = 30 MPa
fy = 410 MPa

Therefore:

$$A_c = \frac{\pi d^2}{4}$$

$$= \frac{\pi \times 400^2}{4}$$

$$= 125.7 \times 10^3 \text{ mm2}$$

$$d_{iax'} = d_{ia} - d'$$

$$= 400 - 50$$

$$= 350.000 \text{ mm}$$

$$d_{iay'} = d_{ia} - d'$$

$$= 400 - 50$$

$$= 350.000 \text{ mm}$$

Assumptions:
(1) The general conditions of clause 3.8.1 are applicable.
(2) The section is symmetrically reinforced.
(3) The specified design axial loads include the self-weight of the column.
(4) 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 steel is chosen.
(3) The corresponding slenderness moments are calculated.
(4) The design axis and design ultimate moment is determined .
(5) The steel required for the design axial force and moment is read from the relevant design chart.
(6) The procedure is repeated until the convergence of the area steel about the design axis.
(7) The area steel perpendicular to the design axis is read from the relevant design chart.

Check column slenderness:

End fixity and bracing for bending about the X-X axis:
The column is unbraced.
$\therefore$ ßx = 2.20 *Table 3.20*

End fixity and bracing for bending about the Y-Y axis:
The column is unbraced.
$\therefore$ ßy = 2.20 *Table 3.20*

Effective column height:

$$l_{ex} = \beta_x \cdot L_o$$
$$= 2.2 \times 7.6$$
$$= 16.720 \text{ m}$$

$$l_{ey} = \beta_y \cdot L_o$$
$$= 2.2 \times 7.6$$
$$= 16.720 \text{ m}$$

Column slenderness about both axes:

$$\lambda_x = \frac{l_{ex}}{d_{ia}}$$
$$= \frac{16.72}{.4}$$
$$= 41.800$$

$$\lambda_y = \frac{l_{ey}}{d_{ia}}$$
$$= \frac{16.72}{.4}$$
$$= 41.800$$

Minimum Moments for Design:

Check for mininum eccentricity: *3.8.2.4*

Check that the eccentricity exceeds the minimum in the plane of bending:

$$e_{min} = 0.05 \cdot d$$
$$= 0.05 \times .4$$
$$= 0.0200 \text{ m}$$

$$M_{min} = e_{min}\ N$$
$$= .02 \times 0$$
$$= 0.0000 \times 10^{0} \text{ kNm}$$

Check if the column is slender: *3.8.1.3*

$\lambda x = 41.8 > 10$

$\lambda y = 41.8 > 10$

$\therefore$ The column is slender.

Check slenderness limit: *3.8.1.8*

Lo = 7.600 mComparision error60dia' = 24.000

$\therefore$ Slenderness limit not exceeded.

Initial moments:

The initial end moments about the X-X axis:
M1 = Smaller initial end moment = 0.0 kNm
M2 = Larger initial end moment = 60.0 kNm

The initial moment near mid-height of the column : *3.8.3.7*

$$M_i = -0.4 \cdot M_1 + 0.6 \cdot M_2$$
$$= -0.4 \times 0 + 0.6 \times 60$$
$$= 36.000 \text{ kNm}$$

$$M_{i2} = 0.4 \cdot M_2$$
$$= 0.4 \times 60$$
$$= 24.000 \text{ kNm}$$

∴ Mi 3 0.4M2 = 36.0 kNm

The initial end moments about the Y-Y axis:
M1 = Smaller initial end moment = 0.0 kNm
M2 = Larger initial end moment = 0.0 kNm

The initial moment near mid-height of the column : *3.8.3.7*

$$M_i = -0.4 \cdot M_1 + 0.6 \cdot M_2$$
$$= -0.4 \times 0 + 0.6 \times 0$$
$$= 0.0000 \times 10^0 \text{ kNm}$$

$$M_{i2} = 0.4 \cdot M_2$$
$$= 0.4 \times 0$$
$$= 0.0000 \times 10^0 \text{ kNm}$$

∴ Mi 3 0.4M2 = 0.0 kNm

Deflection induced moments:

3.8.3.1

Design ultimate capacity of section under axial load only:

$$N_{uz} = 0.4444 \cdot f_{cu} A_c + \frac{1}{1.15} \cdot f_y \cdot A_{sc}$$
$$= 0.4444 \times 30000 \times .12566 + \frac{1}{1.15} \times 410000 \times .00142$$
$$= 2\,181.560 \text{ kN}$$

Maximum allowable stress and strain:

Allowable compression stress in steel

$$f_{sc} = \frac{1}{1.15} \cdot f_y$$
$$= \frac{1}{1.15} \times 410$$
$$= 356.522 \text{ MPa}$$

Allowable tensile stress in steel

$$f_{st} = \frac{1}{1.15} \cdot f_y$$
$$= \frac{1}{1.15} \times 410$$
$$= 356.522 \text{ MPa}$$

Allowable tensile strain in steel

$$e_y = \frac{f_{st}}{E_s}$$
$$= \frac{356.52}{200000}$$
$$= 0.0018$$

Allowable compressive strain in concrete

$$e_c = 0.0035$$

Design ultimate load and moment:

Design axial load:
$P_u = 0.0$ kN

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

(a) 3.8.3.2

$$M = M_2 + M_{add}$$
$$= 60 + 0$$
$$= 60.000 \text{ kNm}$$

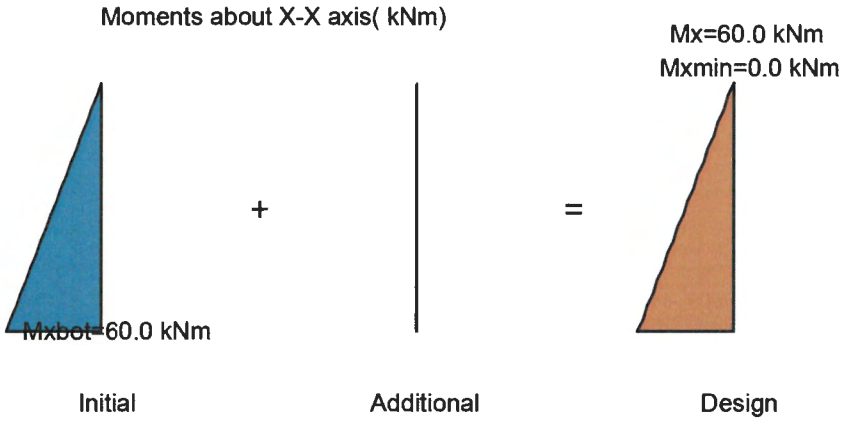
(d) 3.8.3.2

$$M = e_{min} N$$
$$= .02 \times 0$$
$$= 0.0000 \times 10^0 \text{ kNm}$$

Thus 3.8.3.2

M = 60.0 kNm

Moment distribution along the height of the column for bending about the X-X:
At the top, Mx = 0.0 kNm
Near mid-height, Mx = 36.0 kNm
At the bottom, Mx = 60.0 kNm



For bending about the Y-Y axis, the maximum design moment is the greatest of: 3.8.3.7
(a) 3.8.3.2

$M = M_2 + M_{add}$
$= 0 + 0$
$= 0.0000 \times 10^0$ kNm

(d) 3.8.3.2

$M = e_{min}\ N$
$= .02 \times 0$
$= 0.0000 \times 10^0$ kNm

Thus 3.8.3.2

M = 0.0 kNm

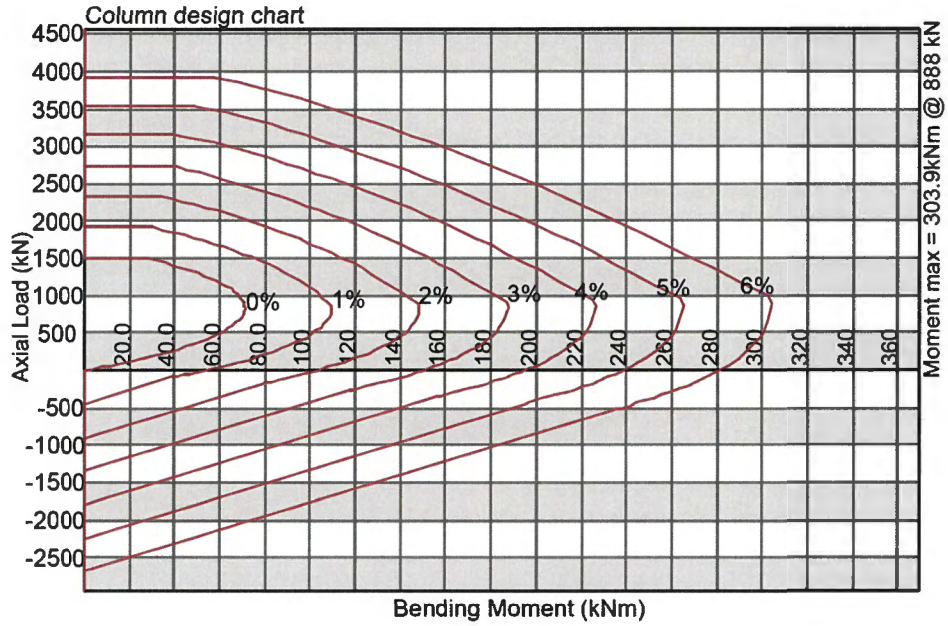
Design of column section for ULS:

Through inspection:
The critical section lies at the bottom end of the column.

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	Job Title	Grusovje6 - armatura pilotne stene	
	Client	Obcina Oplotnica	
	Calcs by	Checked by	Date 11. 03. 2020

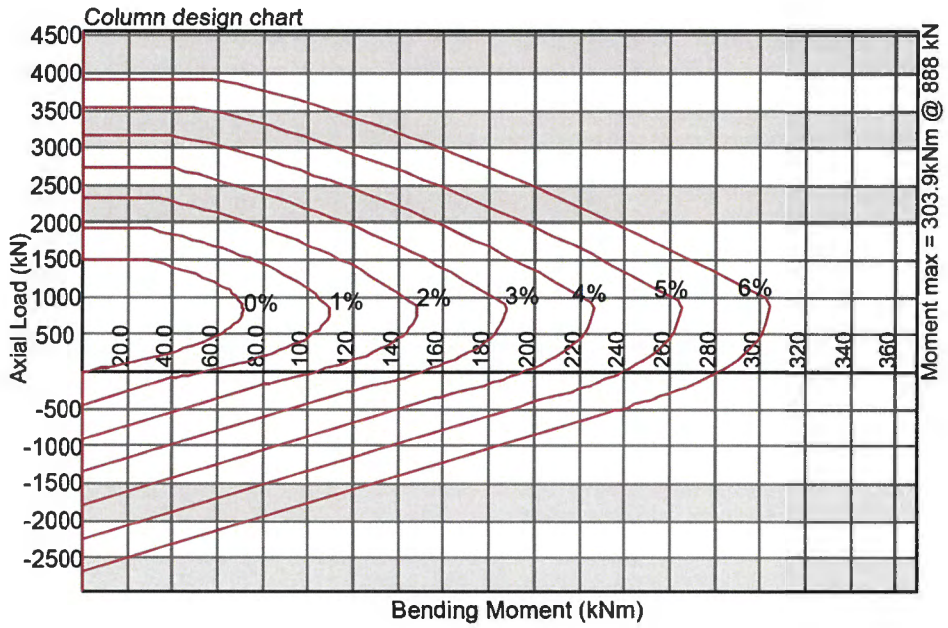
The column is designed to withstand the uni-axially applied moment about the major axis.

For bending about the design axis:



Reinforcement required about the X-X axis:
From the design chart, Asc = 1416 = 1.13%

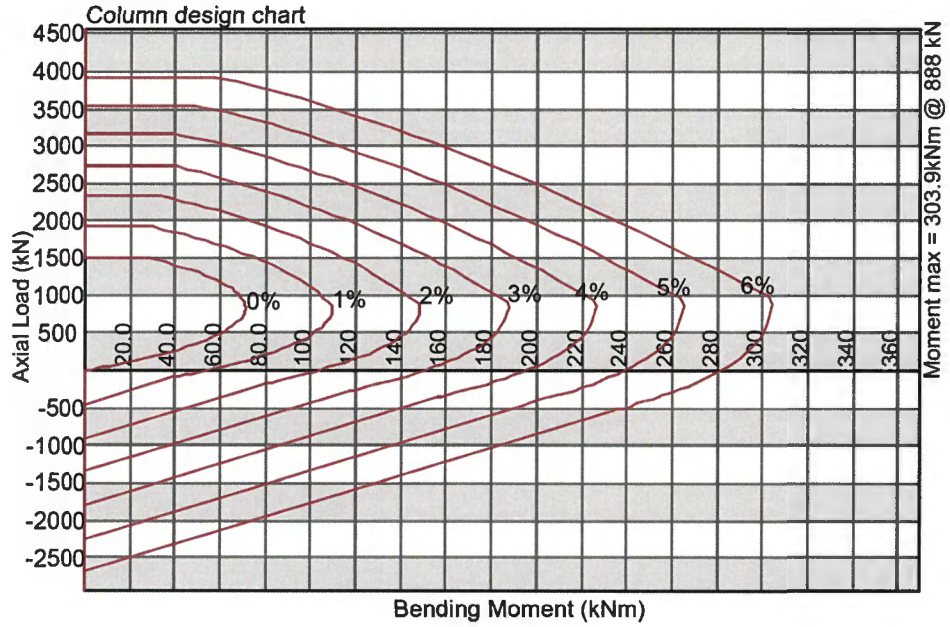
For bending perpendicular to the design axis:



Reinforcement required about the Y-Y axis:
From the design chart, Asc = 503

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	Calcs by	Checked by	Date: 11. 03. 2020

Design chart for bending about any axis:



Summary of design calculations:

Design results for all load cases:

Load case	Axis	N (kN)	M1 (kNm)	M2 (kNm)	Mi (kNm)	Madd (kNm)	Design	M (kNm)	M' (kNm)	Asc (mm2)
1	X-X Y-Y	0.0	0.0 0.0	60.0 0.0	36.0 0.0	0.0 0.0	X-X Bottom	60.0 0.0	60.0	1416 (1.13%)