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ProtaStructure Design Guide

Column Design to BS 8110-1-1997

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Table of Contents

Introduction

By default, ProtaStructure designs columns bent about a single axis, or bent about both axes using the code clauses given in BS 8110-1:1997: Part 1 Section 3.8.

The following table summarises the various stages of the BS8110 column design process:

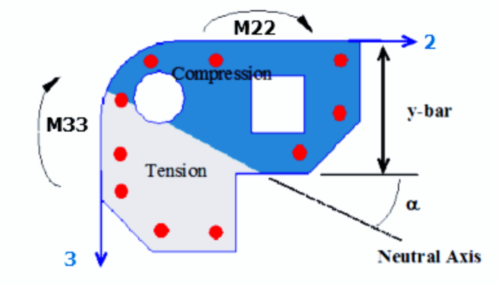
Step	Calculation	Clause
1	Braced or unbraced?	3.8.1.5
2	Calculate effective height using Part 2 of the code	3.8.1.6
3	Check slenderness	3.8.1.3
4	Classify as short or slender	3.8.1.3
5	If slender - calculate M_{add}	3.8.3.1
6	Calculate minimum moments	3.8.2.4
7a	If braced - calculate design moments	3.8.3.2
7b	If unbraced - calculate design moments	3.8.3.7
8	If using the BS8110 design method -Calculate equivalent uni-axial design moments *(If using the Bi-Axial design method -skip this stage)*	3.8.4.5
9	Member Design	3.8.4

As indicated in the table, the program provides two design methods.

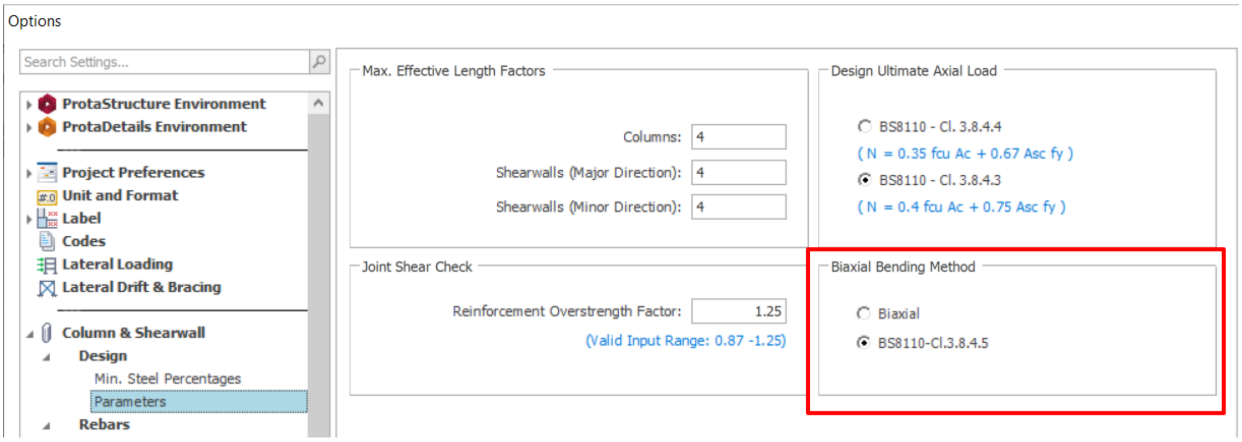
The default method applies Cl 3.8.4.5 to convert bi-axial moments into an equivalent uni-axial design moment.

Alternatively, if the bi-axial design method is selected, the bi-axial moments calculated in step 7 are fed directly into the member design stage and a more rigorous solution technique developed from first principles is adopted. This can produce some economy, however because the neutral axis will lie on an incline the results of the design process will be more difficult to cross check.

For poly-line columns, as shown below, the bi-axial design method will always be adopted.



The choice of design method is set via the ***Column & Shearwall Parameters settings*** dialog shown below.



Max. Effective Length Factors

Columns: 4

Shearwalls (Major Direction): 4

Shearwalls (Minor Direction): 4

Design Ultimate Axial Load

○ BS8110 - Cl. 3.8.4.4

(N = 0.35 fcu Ac + 0.67 Asc fy)

● BS8110 - Cl. 3.8.4.3

(N = 0.4 fcu Ac + 0.75 Asc fy)

Joint Shear Check

Reinforcement Overstrength Factor: 1.25

(Valid Input Range: 0.87 -1.25)

Biaxial Bending Method

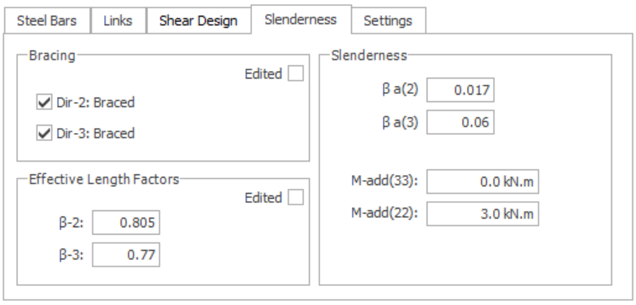
○ Biaxial

● BS8110-Cl.3.8.4.5

The BS8110 Column Design Process

1. Braced or unbraced - Cl 3.8.1.5

Globally, columns will be considered as braced if this option has been selected in the Building Parameters. Individual columns can have their braced/unbraced status modified within the Column Interactive Design, via the Slenderness tab as shown below:



Bracing

Edited ☐

☑ Dir-2: Braced

☑ Dir-3: Braced

Effective Length Factors

Edited ☐

β-2: 0.805

β-3: 0.77

Slenderness

β a(2) 0.017

β a(3) 0.06

M-add(33): 0.0 kN.m

M-add(22): 3.0 kN.m

For walls, the braced/unbraced status is applied in the same way as it is for columns. However, it should be noted that walls can always be considered as braced along their major axis (i.e dir 1).

2. Calculate effective height - Cl 3.8.1.6

The effective height is determined from the equation:

$$l_e = \beta l_o$$

A rigorous assessment of the effective length is undertaken using the formulae given in **2.5 of BS 8110-2:1985.** Perhaps surprisingly, this can often result in a greater effective length than is determined from the **Tables 3.19** and **3.20** of **BS 8110-1:1997.**

The beta value determined by part 2 can be edited and replaced by the value from the tables if required.

3. Check slenderness limits - Cl 3.8.1.7 & 3.8.1.8

The slenderness limits for columns, l_o, should not exceed 60 times the smaller dimensions of a column. However, the slenderness limit of unbraced columns, l_o, should satisfy the followings:

$$l_o \leq 60b \; or \; \frac{100b^2}{h}; whichever \; is \; less. \qquad equation \; 31$$

In equation 31, *h* and *b* are respectively the larger and smaller dimensions of the column.

4. Classify as short or slender- Cl 3.8.1.3

Columns and walls are considered as short when both the ratios l_{ex}/h and l_{ey} / b are less than 15 (braced) and 10 (unbraced), otherwise they are slender.

5. If slender - calculate M_add- Cl 3.8.3.1

In order to calculate the additional moment induced in the column it is required that factor K be determined. Although the code allows for K to be conservatively taken as 1.0, ProtaStructure calculates K using the equation 33 in the code:

$$K = \frac{N_{uz} - N}{N_{uz} - N_{bal}} \leq 1 \qquad equation \; 33$$

The calculation of $As_{required}$ is itself an iterative process and K is re-calculated at every iteration.

The following assumptions are applied to ensure the calculation of K remains slightly conservative.

<u>Calculation of N_{uz}:</u>

$$N_{uz} = 0.45 f_{cu} A_c + 0.87 f_y A_{sc}$$

Equation has two parts:

- Steel $(0.87 f_y A_{sc})$ - In this equation, ProtaStructure uses A_{sc} required. (since it is logical that we should be able to fail a section by providing more steel than it is required)
- Concrete $(0.45 f_{cu} A_c)$ – The net concrete is used in this equation.

<u>Calculation of N_{bal}:</u>

The code indicates that this is based on $0.25f_{cu}bd$ (d = eff depth). However, with the aim of keeping N_{bal} large (hence making the calculation of K more conservative), actually uses the gross concrete area here.

6. Calculate minimum moments - Cl 3.8.2.4

The minimum design moment is calculated in both directions taking the design ultimate axial load acting at a minimum eccentricity as per Cl 3.8.2.4.

7. Design Moments

a. If braced, calculate design moments about each axis – Cl 3.8.3.2

The design moment is calculated in both directions as the greatest of:

i. M_2;
ii. M_i+M_{add};
iii. $M_1+M_{add}/2$;
iv. $e_{min}N$.

where M_1, M_2 and M_{add} are as defined in Figure 3.20.
$M_i = 0.4\ M_1 + 0.6\ M_2 \geq 0.4\ M_2$

b. If unbraced, calculate design moments about each axis – Cl 3.8.3.2

The design moment is calculated in both directions as per Figure 3.21.

8. Calculate equivalent uni-axial design moments - Cl 3.8.4.5

Because there will always be at least a minimum moment acting in both directions, for rectangular columns the design moment will always be determined from Equations 40 and 41 in the code.

$$For\ \ M_x/h'\ \geq\ M_y/b'\,, M_x' = M_x + \beta\frac{h'}{b'}M_y \qquad\qquad Equation\ 40$$

$$For\ \ M_x/h'\ \geq\ M_y/b'\,, M_y' = M_y + \beta\frac{h'}{b'}M_x \qquad\qquad Equation\ 41$$

Where *h'* and *b'* are shown in Figure 3.22;
b is the coefficient obtained from Table 3.22;

For circular columns, the moments in the two directions are resolved.

$$M = \sqrt{{M_x}^2 + {M_y}^2}$$

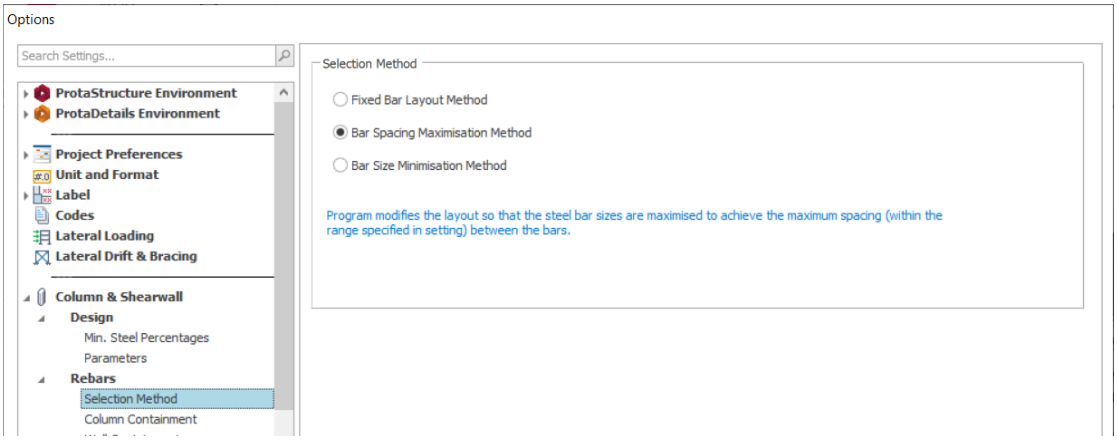
9. Member Design – Cl 3.8.4

With the design axial load and design moment established, the program determines the required steel area using the BS8110 stress block. The neutral axis of the cross section is determined and a bar size and spacing obtained to provide sufficient moment capacity.

Each design combination is considered and the one that results in the largest steel area requirement is selected as being critical.

If the minimum area of steel is satisfactory for every combination, the program will record combination 1 as being critical, (irrespective of the relative magnitude of loads in each combinations).

Three methods of bar selection are available:



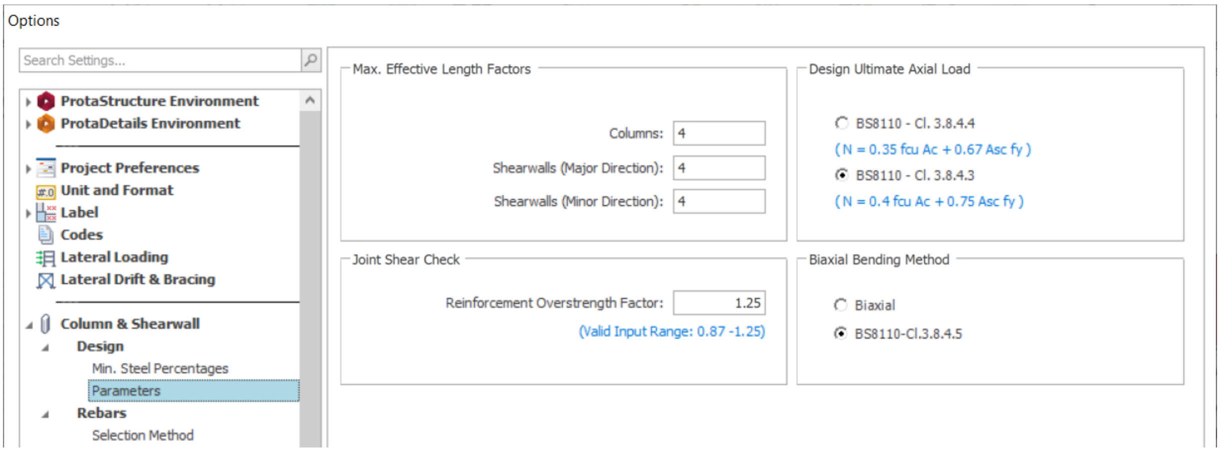
Selection Method

○ Fixed Bar Layout Method
● Bar Spacing Maximisation Method
○ Bar Size Minimisation Method

Program modifies the layout so that the steel bar sizes are maximised to achieve the maximum spacing (within the range specified in setting) between the bars.

- Fixed bar layout — The bar locations are defined by the user and the program determines the bar size required.
- Bar Spacing Maximisation — The program determines the bar size and spacing with the aim to maximise the spacing. This is normally the preferred option.
- Bar Size Minimisation — The program determines the bar size and spacing with the aim to minimise the bar size.

The maximum axial load is checked against **Cl 3.8.4.3** or **Cl 3.8.4.4.** The program defaults to the more conservative capacity determined by **Cl 3.8.4.3.** The clause used can be changed via the BS8110 tab of the column design settings as shown:



Max. Effective Length Factors

Columns: 4
Shearwalls (Major Direction): 4
Shearwalls (Minor Direction): 4

Design Ultimate Axial Load

○ BS8110 - Cl. 3.8.4.4
(N = 0.35 fcu Ac + 0.67 Asc fy)
● BS8110 - Cl. 3.8.4.3
(N = 0.4 fcu Ac + 0.75 Asc fy)

Joint Shear Check

Reinforcement Overstrength Factor: 1.25
(Valid Input Range: 0.87 -1.25)

Biaxial Bending Method

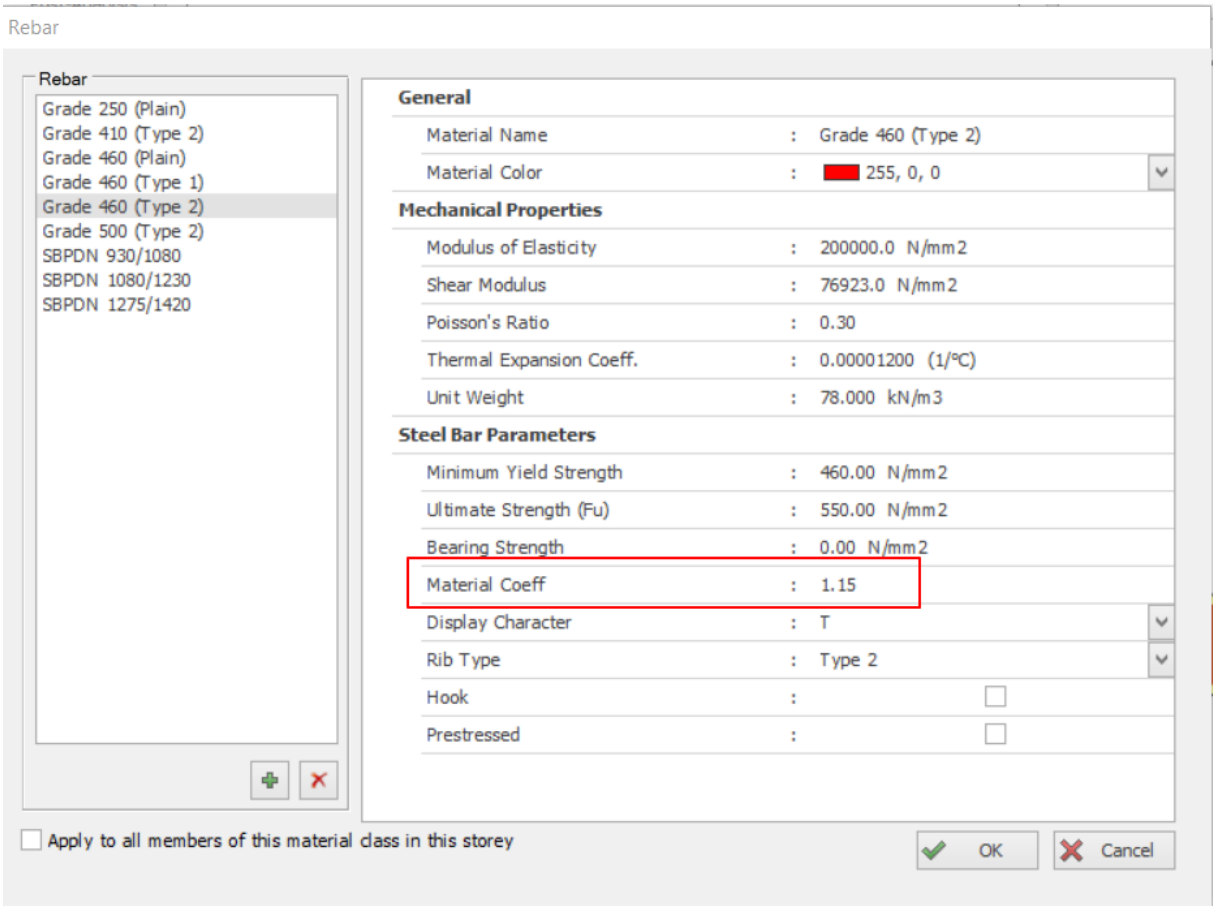
○ Biaxial
● BS8110-Cl.3.8.4.5

Worked Examples

The Design Model

The example model *Doc_Example_4* is opened and saved to a new name (so as not to destroy the original example). The copied model is then adjusted so that its storey height is increased to 5.5m and it is then re-analysed. In this model the steel grade is 460 and the steel material factor is 1.15.

The value of steel material factor is taken from the BS 8110-1-1997: Table 2.2. The value of steel material factor can be changed in the Rebar Properties if the users wish to overwrite the default value.



General

Material Name : Grade 460 (Type 2)

Material Color : 🟥 255, 0, 0

Mechanical Properties

Modulus of Elasticity : 200000.0 N/mm2

Shear Modulus : 76923.0 N/mm2

Poisson's Ratio : 0.30

Thermal Expansion Coeff. : 0.00001200 (1/°C)

Unit Weight : 78.000 kN/m3

Steel Bar Parameters

Minimum Yield Strength : 460.00 N/mm2

Ultimate Strength (Fu) : 550.00 N/mm2

Bearing Strength : 0.00 N/mm2

Material Coeff : 1.15

Display Character : T

Rib Type : Type 2

Hook : ☐

Prestressed : ☐

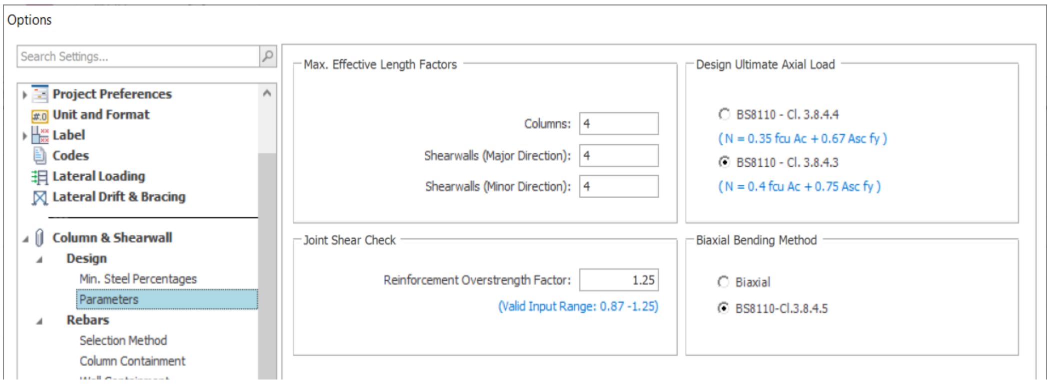
☐ Apply to all members of this material class in this storey

Bar diameters of 12mm are used. The minimum link diameter is set to 10mm. Note however, the actual design process would be identical irrespective of which steel grade and material factor and bar diameters are used.

Column Design Settings

The column design settings initially adopted are as shown:

Design Parameters:



Max. Effective Length Factors

Columns: 4

Shearwalls (Major Direction): 4

Shearwalls (Minor Direction): 4

Design Ultimate Axial Load

○ BS8110 - Cl. 3.8.4.4 (N = 0.35 fcu Ac + 0.67 Asc fy)

● BS8110 - Cl. 3.8.4.3 (N = 0.4 fcu Ac + 0.75 Asc fy)

Joint Shear Check

Reinforcement Overstrength Factor: 1.25

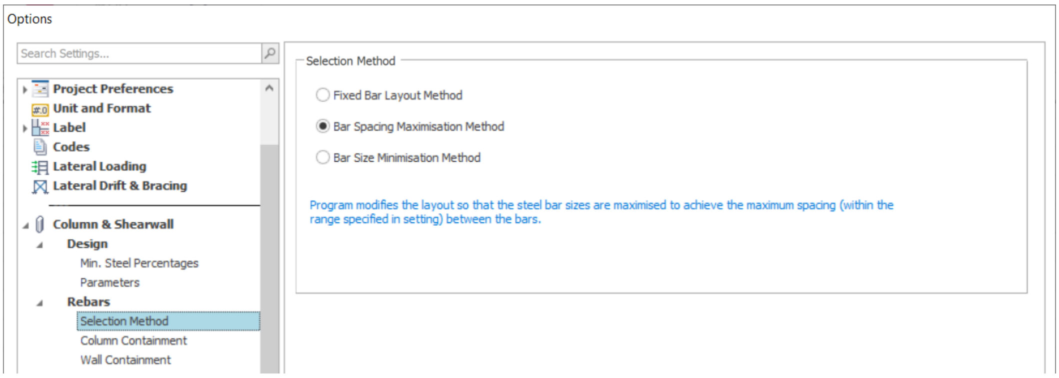
(Valid Input Range: 0.87 -1.25)

Biaxial Bending Method

○ Biaxial

● BS8110-Cl.3.8.4.5

Steel Bars – Layout/Selection:



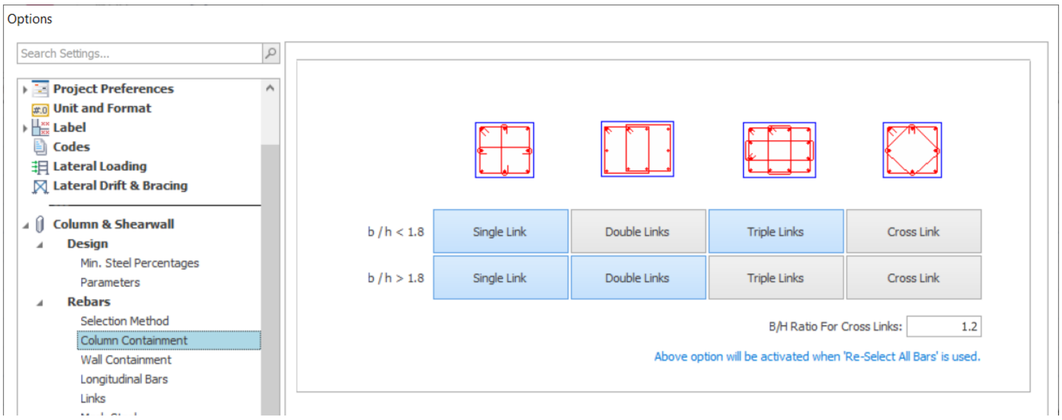
Selection Method

○ Fixed Bar Layout Method

● Bar Spacing Maximisation Method

○ Bar Size Minimisation Method

Program modifies the layout so that the steel bar sizes are maximised to achieve the maximum spacing (within the range specified in setting) between the bars.

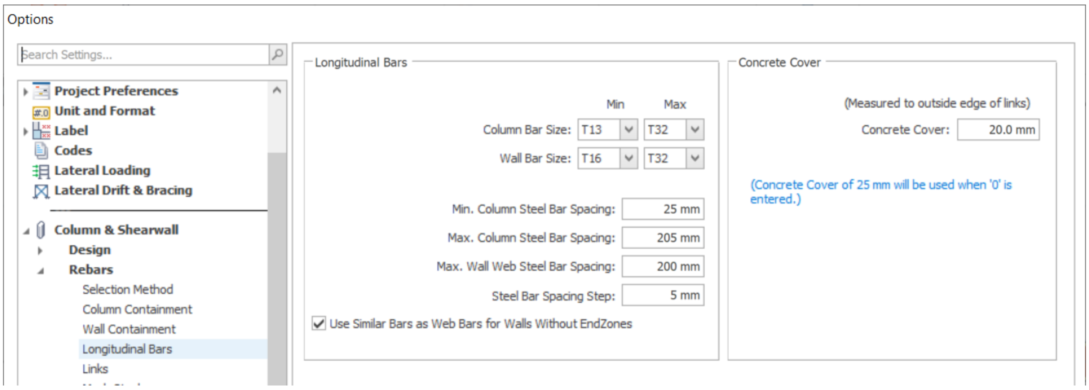


b / h < 1.8	Single Link	Double Links	Triple Links	Cross Link
b / h > 1.8	Single Link	Double Links	Triple Links	Cross Link

B/H Ratio For Cross Links: 1.2

Above option will be activated when 'Re-Select All Bars' is used.

Steel Bars – Longitudinal Steel:



Longitudinal Bars

	Min	Max
Column Bar Size:	T13	T32
Wall Bar Size:	T16	T32

Min. Column Steel Bar Spacing: 25 mm

Max. Column Steel Bar Spacing: 205 mm

Max. Wall Web Steel Bar Spacing: 200 mm

Steel Bar Spacing Step: 5 mm

☑ Use Similar Bars as Web Bars for Walls Without EndZones

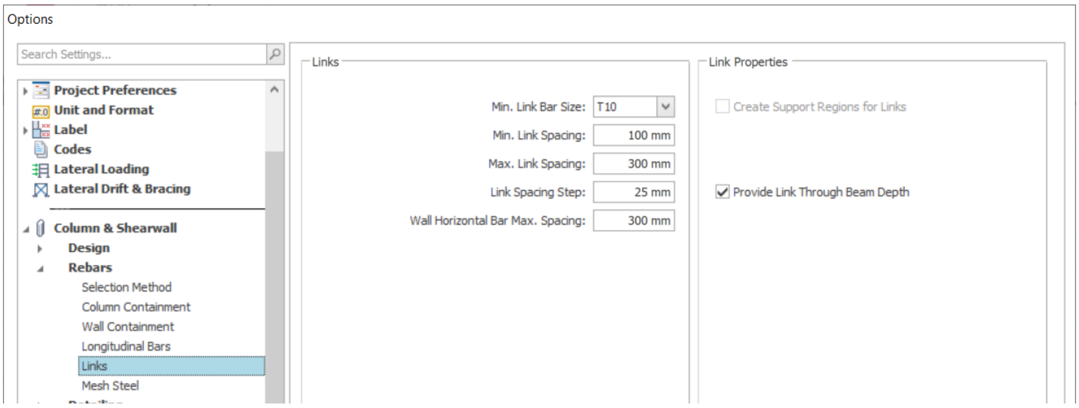
Concrete Cover

(Measured to outside edge of links)

Concrete Cover: 20.0 mm

(Concrete Cover of 25 mm will be used when '0' is entered.)

Steel Bars – Links:



Links

Min. Link Bar Size: T10

Min. Link Spacing: 100 mm

Max. Link Spacing: 300 mm

Link Spacing Step: 25 mm

Wall Horizontal Bar Max. Spacing: 300 mm

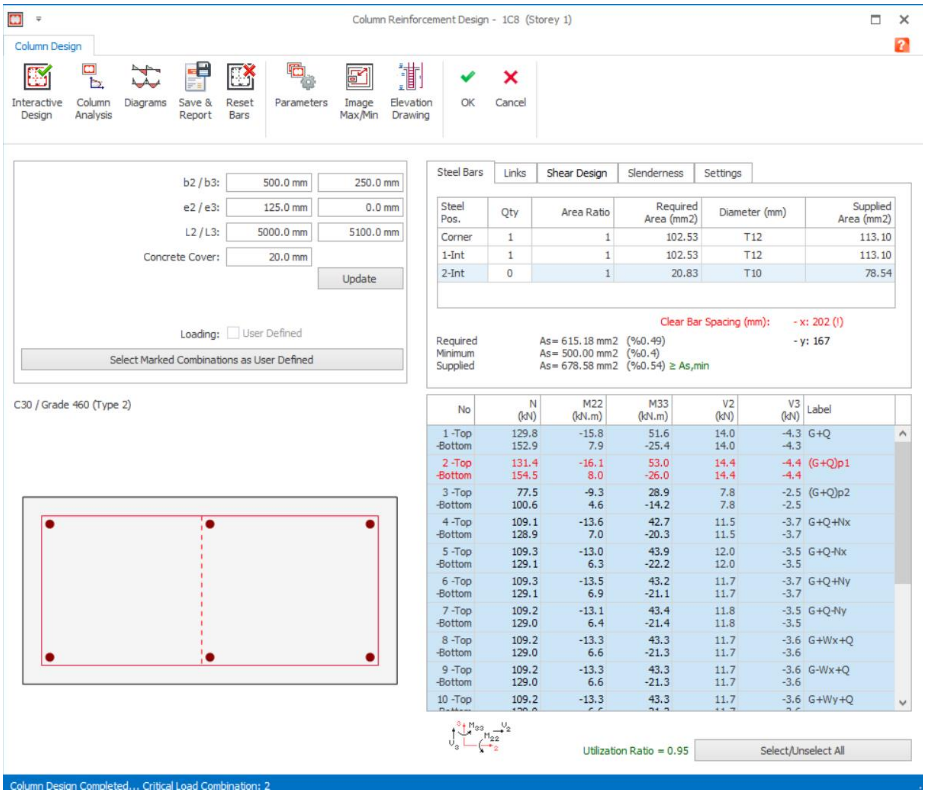
Link Properties

☐ Create Support Regions for Links

☑ Provide Link Through Beam Depth

Braced Rectangular Column Example

Column 1C8 will be used to demonstrate the design process for a rectangular column.



b2 / b3: 500.0 mm 250.0 mm
e2 / e3: 125.0 mm 0.0 mm
L2 / L3: 5000.0 mm 5100.0 mm
Concrete Cover: 20.0 mm
Loading: ☐ User Defined

C30 / Grade 460 (Type 2)

Steel Pos.	Qty	Area Ratio	Required Area (mm2)	Diameter (mm)	Supplied Area (mm2)
Corner	1	1	102.53	T12	113. 10
1-Int	1	1	102.53	T12	113. 10
2-Int	0	1	20.83	T10	78.54

Clear Bar Spacing (mm): - x: 202 (!)
- y: 167

Required As= 615. 18 mm2 (%0.49)
Minimum As= 500.00 mm2 (%0.4)
Supplied As= 678.58 mm2 (%0.54) ≥ As,min

No	N (kN)	M22 (kN.m)	M33 (kN.m)	V2 (kN)	V3 (kN)	Label
1 -Top -Bottom	129.8 152.9	-15.8 7.9	51.6 -25.4	14.0 14.0	-4.3 -4.3	G+Q
2 -Top -Bottom	131.4 154.5	-16.1 8.0	53.0 -26.0	14.4 14.4	-4.4 -4.4	(G+Q)p1
3 -Top -Bottom	77.5 100.6	-9.3 4.6	28.9 -14.2	7.8 7.8	-2.5 -2.5	(G+Q)p2
4 -Top -Bottom	109.1 128.9	-13.6 7.0	42.7 -20.3	11.5 11.5	-3.7 -3.7	G+Q+Nx
5 -Top -Bottom	109.3 129.1	-13.0 6.3	43.9 -22.2	12.0 12.0	-3.5 -3.5	G+Q-Nx
6 -Top -Bottom	109.3 129.1	-13.5 6.9	43.2 -21.1	11.7 11.7	-3.7 -3.7	G+Q+Ny
7 -Top -Bottom	109.2 129.0	-13.1 6.4	43.4 -21.4	11.8 11.8	-3.5 -3.5	G+Q-Ny
8 -Top -Bottom	109.2 129.0	-13.3 6.6	43.3 -21.3	11.7 11.7	-3.6 -3.6	G+Wx+Q
9 -Top -Bottom	109.2 129.0	-13.3 6.6	43.3 -21.3	11.7 11.7	-3.6 -3.6	G-Wx+Q
10 -Top	109.2	-13.3	43.3	11.7	-3.6	G+Wy+Q

Utilization Ratio = 0.95

Column Design Completed... Critical Load Combination: 2

- Column dimension in direction 2, b2 = 500 mm
- column dimension in direction 3, b3 = 250 mm

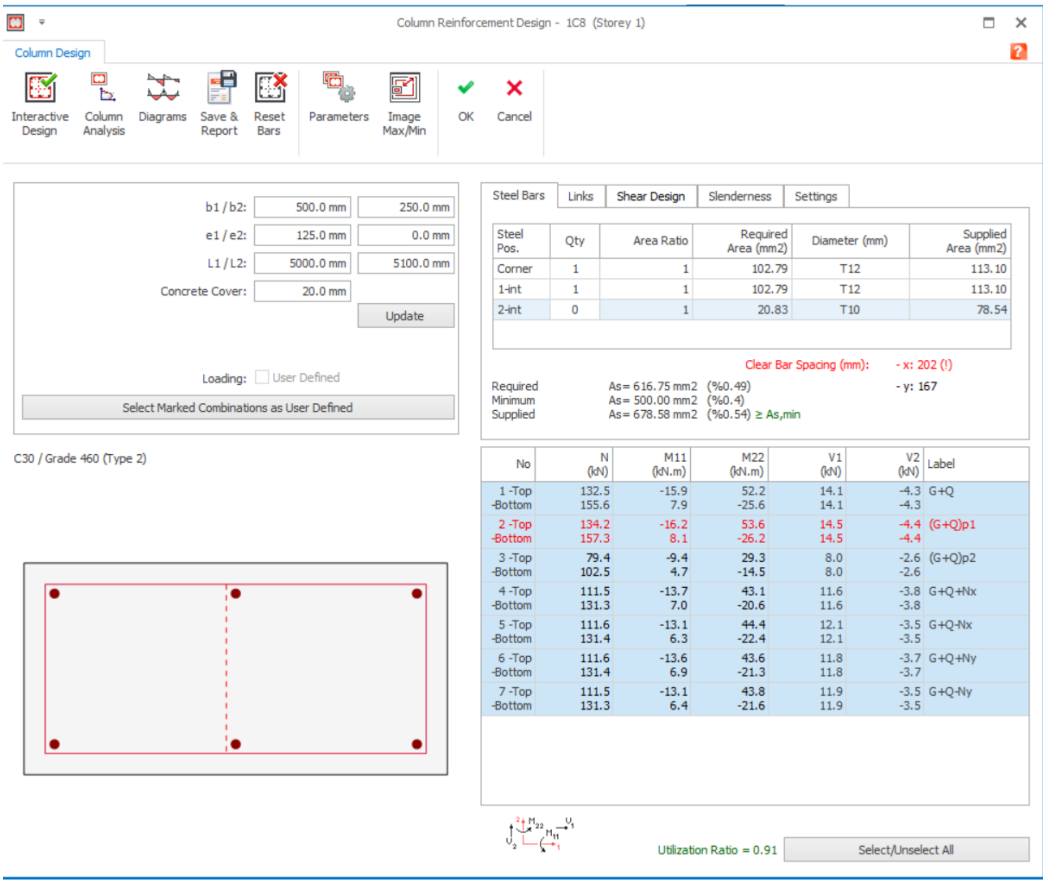
The clear height of column in the two directions takes account of the beams framing into the top of the column.

- L_o2 = 5500mm - 500mm = 5000 mm
- L_o3 = 5500mm - 400mm = 5100 mm

As shown on the design screen above, if only 3 bars are placed in the x direction the default clear bar spacing limit of 200 mm (as specified in the Column Design Settings) would be slightly exceeded. In the worked examples the Max. Column Steel Bar Spacing has been relaxed to 205mm in order that the above bar layout can be used.

Performing the Design

Click the Design button to perform the calculations. This will design the column for all the highlighted design combinations. Design combination 2 is found to be critical and is highlighted in red in the table as shown below.



b1 / b2: 500.0 mm 250.0 mm
e1 / e2: 125.0 mm 0.0 mm
L1 / L2: 5000.0 mm 5100.0 mm
Concrete Cover: 20.0 mm
Loading: ☐ User Defined

C30 / Grade 460 (Type 2)

Steel Pos.	Qty	Area Ratio	Required Area (mm2)	Diameter (mm)	Supplied Area (mm2)
Corner	1	1	102.79	T12	113.10
1-int	1	1	102.79	T12	113.10
2-int	0	1	20.83	T10	78.54

Clear Bar Spacing (mm): - x: 202 (!)
- y: 167

Required As= 616.75 mm2 (%0.49)
Minimum As= 500.00 mm2 (%0.4)
Supplied As= 678.58 mm2 (%0.54) ≥ As,min

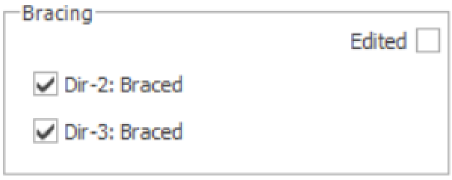
No	N (kN)	M11 (kN.m)	M22 (kN.m)	V1 (kN)	V2 (kN)	Label
1 -Top -Bottom	132.5 155.6	-15.9 7.9	52.2 -25.6	14.1 14.1	-4.3 -4.3	G+Q
2 -Top -Bottom	134.2 157.3	-16.2 8.1	53.6 -26.2	14.5 14.5	-4.4 -4.4	(G+Q)p1
3 -Top -Bottom	79.4 102.5	-9.4 4.7	29.3 -14.5	8.0 8.0	-2.6 -2.6	(G+Q)p2
4 -Top -Bottom	111.5 131.3	-13.7 7.0	43.1 -20.6	11.6 11.6	-3.8 -3.8	G+Q+Nx
5 -Top -Bottom	111.6 131.4	-13.1 6.3	44.4 -22.4	12.1 12.1	-3.5 -3.5	G+Q-Nx
6 -Top -Bottom	111.6 131.4	-13.6 6.9	43.6 -21.3	11.8 11.8	-3.7 -3.7	G+Q+Ny
7 -Top -Bottom	111.5 131.3	-13.1 6.4	43.8 -21.6	11.9 11.9	-3.5 -3.5	G+Q-Ny

Utilization Ratio = 0.91

Each stage of this design process will now be examined in detail.

1. Braced or unbraced – Cl 3.8.1.5

In this example the column has been defined as braced in both directions. This can be confirmed by clicking on the Slenderness tab.



Bracing

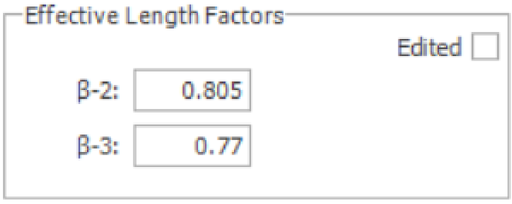
Edited ☐

☑ Dir-2: Braced

☑ Dir-3: Braced

2. Calculate effective height – Cl 3.8.1.6

The effective length factors 2 and 3 that have been calculated are also displayed on the Slenderness tab as shown.



Effective Length Factors

Edited ☐

β-2: 0.805

β-3: 0.77

These effective length factors are calculated as follows:

In Direction 2:

Beam stiffness at top of the column

L = 5500 mm, b = 250 mm, d = 500 mm

$k_{b1} = b \times d^3/(12 \times L) = 473484.85\ mm^3$

Column stiffness

$k_{c1} = b_2 \times {b_1}^3/(12 \times L_{o1}) = 520833.33\ mm^3$

Calculation using the formulae given in **BS 8110-2:1985 Cl 2.5**

$\alpha_{c,2} = \frac{k_{c1}}{k_{b1}} = 1.10$

$\alpha_{c,1} = 1.0(Fixed\ based\ is\ defined\ in\ this\ example)$

$\alpha_{c,min} = 1.0\ (lesser\ of\ \alpha_{c,2}\ or\ \ \alpha_{c,1})$

$Equation\ 3\quad Effective\ Length\ Factors, \beta = [0.7 + 0.05(\alpha_{c,1} + \alpha_{c,2})] = \mathbf{0.805}$

$Equation\ 4\quad Effective\ Length\ Factors, \beta = [0.85 + 0.05(\alpha_{c,min})] = \mathbf{0.9}$

$\beta = \mathbf{0.805}\ (\boldsymbol{whichever\ is\ lesser})$

In Direction 3:

Beam stiffness at top of the column

L = 4250 mm, b = 250 mm, d = 400 mm

$k_{b1} = b \times d^3/(12 \times L) = 313725.49\ mm^3$

Column stiffness

$k_{c1} = b_1 \times {b_2}^3/(12 \times L_{o2}) = 127655.23\ mm^3$

Calculation using the formulae given in **BS 8110-2:1985 Cl 2.5**

$\alpha_{c,2} = \frac{k_{c1}}{k_{b1}} = 0.407$

$\alpha_{c,1} = 1.0(Fixed\ based\ is\ defined\ in\ this\ example)$

$\alpha_{c,min} = 0.407\ (lesser\ of\ \alpha_{c,2}\ or\ \ \alpha_{c,1})$

$Equation\ 3\quad Effective\ Length\ Factors, \beta = [0.7 + 0.05(\alpha_{c,1} + \alpha_{c,2})] = \mathbf{0.770}$

$Equation\ 4\quad Effective\ Length\ Factors, \beta = [0.85 + 0.05(\alpha_{c,min})] = \mathbf{0.87}$

$\beta = \mathbf{0.770}\ (\boldsymbol{whichever\ is\ lesser})$

Effective Member Length

$$l_{e2} = \beta_{dir-2} \times l_{o2} = 4025\ mm$$

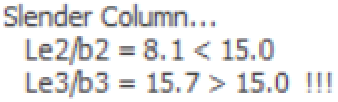
$$l_{e3} = \beta_{dir-3} \times l_{o3} = 3927\ mm$$

3. Check Slenderness limits – Cl 3.8.1.7 & 3.8.1.8

$$l_o \leq 60b = 15000\ mm, ok!$$

4. Classify as short or slender – Cl 3.8.1.3

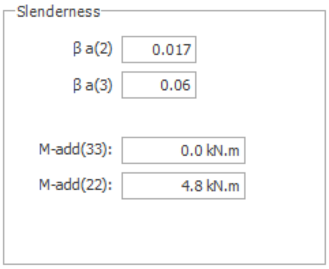
The classification is shown on the Column Reinforcement Design dialog.



Slender Column...
Le2/b2 = 8.1 < 15.0
Le3/b3 = 15.7 > 15.0 !!!

5. If slender – Calculate M_{add} – Cl 3.8.3.1

Depending on the classification the b_a and M_{add} values have been calculated accordingly and are displayed on the Slenderness tab.



Slenderness

β a(2) 0.017

β a(3) 0.06

M-add(33): 0.0 kN.m

M-add(22): 4.8 kN.m

In Direction 2

$$\beta_a = \frac{1}{2000}\left(\frac{l_e}{b'}\right)^2 \qquad Equation\ 34$$

Column is not slender in direction 2, hence $M_{add,33}$ = 0 kNm

In Direction 3

$$\beta_a = \frac{1}{2000}\left(\frac{l_e}{b'}\right)^2 \qquad Equation\ 34$$

$$\beta_a = \frac{1}{2000}\left(\frac{3927}{250}\right)^2 = 0.123$$

Column is slender in direction 3, hence $M_{add,22}$ must be calculated:

$$K = \frac{N_{uz} - N}{N_{uz} - N_{bal}} \leq 1$$

- Applied Axial Load, N= 157.6 kN
- Column Dimension (in direction which under consideration), b_3=500 mm

- Column Width, b_2=250mm
- Concrete Grade, f_{cu}=40 N/mm^2
- Steel Grade, f_y= 460 N/mm^2
- Material factor for steel, s= 1.150
- Area of steel required, As_{req}= 636.08 mm^2

$$N_{uz} = 0.45 f_{cu} A_c + 0.87 f_y A_{sc}$$
$$= 0.45 \times 40 \times \left[(b_2 \times h) - As_{req}\right] + 0.87 \times f_y \times As_{req} = \mathbf{2260.69\ kN}$$

$$N_{bal} = 0.25 f_{cu} bh = \mathbf{1250\ kN}.$$

$$K = \frac{N_{uz} - N}{N_{uz} - N_{bal}} = 2.08, \qquad \textbf{hence K = 1}$$

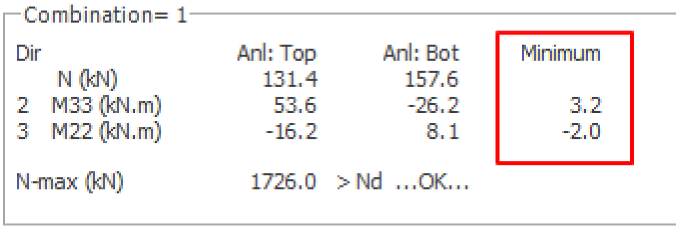
$$\alpha_u = \beta K b_2 = \mathbf{30.75\ mm} \qquad Equation\ 32$$

$$M_{add} = N\alpha_u = \mathbf{4.84\ kNm} \qquad Equation\ 35$$

Hence, M_{add} about direction 2 (in direction - 3) is 4.84 kNm

6. Calculate minimum moments – Cl 3.8.2.4

These are shown on the Column Reinforcement Design dialog.



```
Combination= 1
Dir                  Anl: Top    Anl: Bot     Minimum
     N (kN)            131.4       157.6
2   M33 (kN.m)          53.6       -26.2          3.2
3   M22 (kN.m)         -16.2         8.1         -2.0

N-max (kN)            1726.0   > Nd  ...OK...
```

Minimum eccentricity 2= min (0.05 x h, 20 mm) = 20.0 mm

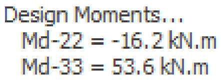
Minimum eccentricity 3= min (0.05 x b, 20 mm) = 12.5 mm

$M_{min,33}$ =N x 20 mm= **3.15 kNm**

$M_{min,22}$ =N x 12.5 mm= **1.97 kNm**

7. Calculate design moments about each axis – Cl 3.8.3.2

These are also shown on the Column Reinforcement Design dialog.



```
Design Moments...
  Md-22 = -16.2 kN.m
  Md-33 = 53.6 kN.m
```

In direction 2 (About direction – 3):

- Smaller end moment, M_1= -26.2 kNm
- Larger end moment, M_2= 53.6 kNm
- M_i = 0.4 M_1+ 0.6 M_2 ≥ 0.4 M_2 = 21.68 kNm ≥ 21.44 kNm
- Hence, M_i = 21.68 kNm

- $M_{d,33\ eff}$ is the greatest of:
  - a) M_2 =53.6 kNm
  - b) $M_i + M_{add}$ = 21.68 kNm
  - c) M_1+M_{add}/2 = 21.68 kNm
  - d) $E_{min}N$ = 3.15 kNm

$M_{d,33\ eff}$ = 53.6 kNm.

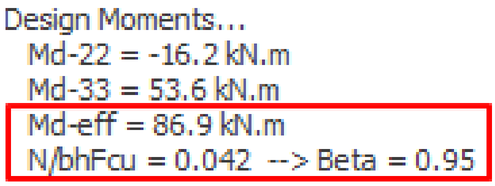
<u>In direction 3 (About direction – 2):</u>

- Smaller end moment, M_1= 8.1 kNm
- Larger end moment, M_2= -16.2 kNm
- $M_i = 0.4\ M_1 + 0.6\ M_2 \geq 0.4\ M_2$ = <u>-6.48 kNm ≥ -6.48 kNm</u>
- Hence, M_i = -6.48 kNm
- $M_{d,22\ eff}$ is the greatest of:
  - a) M_2 = -16.2 kNm
  - b) $M_i + M_{add}$ = -4.56 kNm
  - c) M_1+M_{add}/2 = 9.09 kNm
  - d) $E_{min}N$ = 1.97 kNm

$M_{d,22\ eff}$ = -16.2 kNm.

8. Calculate equivalent uni-axial design moments – Cl 3.8.4.5

The effective design moment is calculated from Equations 40 and 41.



```
Design Moments...
  Md-22 = -16.2 kN.m
  Md-33 = 53.6 kN.m
  Md-eff = 86.9 kN.m
  N/bhFcu = 0.042  --> Beta = 0.95
```

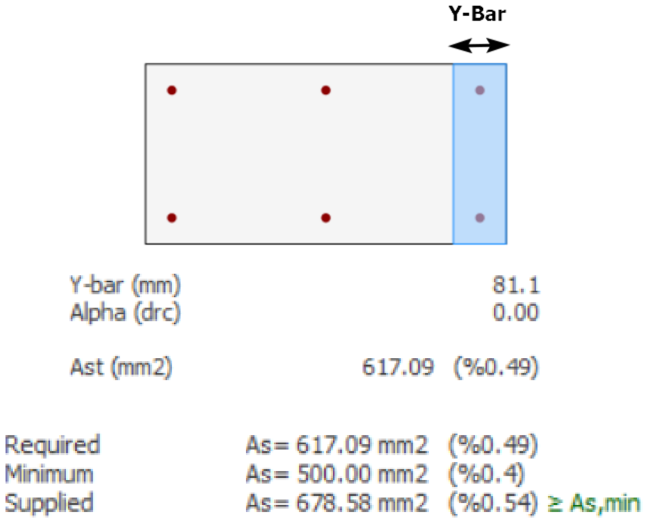
- Longitudinal bar diameter = 12 mm
- Cover = 20mm
- Links = 10mm
- $M_x = M_{d,22\ eff}$ = 16.2 kNm
- $M_y = M_{d,33\ eff}$ = 53.6 kNm
- h' = b_2 – cover – links - diameter/2 = 214 mm
- b' = b_1 – cover – links – diameter/2 = 464 mm
- M_x/h' = 75.7 kN, M_y/b' = 115.52 kN,
- M_x/h' < M_y/b', hence $M'_y = M_y + \beta \frac{b'}{h'} M_x$ $\quad equation\ 41$
- $\frac{N}{bhf_{cu}} = 0.042, \quad hence\ \beta = 0.95 \quad (interpolated\ from\ TABLE\ 3.22)$
- $M'_y = M_y + \beta \frac{b'}{h'} M_x = \ 86.97\ kNm$

9. Member Design – Cl 3.8.4

<u>Design forces:</u>

- N = 151.8 kN
- M_y' = 86.8 kNm
- M_x' = 0 kNm

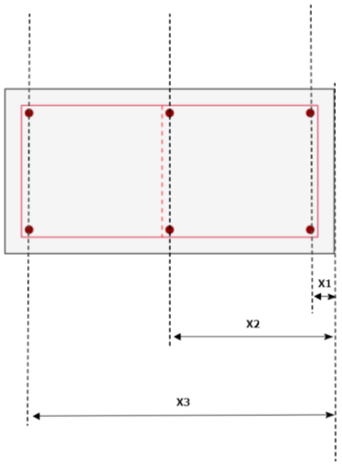
Solution determined by ProtaStructure:



```
Y-bar (mm)                    81.1
Alpha (drc)                   0.00

Ast (mm2)              617.09  (%0.49)

Required        As= 617.09 mm2  (%0.49)
Minimum         As= 500.00 mm2  (%0.4)
Supplied        As= 678.58 mm2  (%0.54) ≥ As,min
```

- Distance to neutral axis-Y bar = 81.1 mm
- Area of steel required, $As_{required}$ = 617.09 mm^2
- Area of steel provided, $As_{provided}$ = 678.58 mm^2

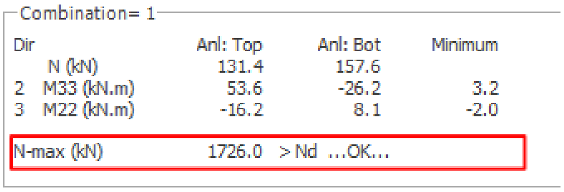
Reinforcement in Section



- X1 = cover + links + diameter/2 = 36 mm
- X2 = 500 mm/2 = 250 mm
- X3 = 500 mm – cover – links - diameter/2 = 464 mm

Y-Bar > X1, hence bars at X1 are in compression.

Calculate maximum axial load – Cl 3.8.4.3 or Cl 3.8.4.4:



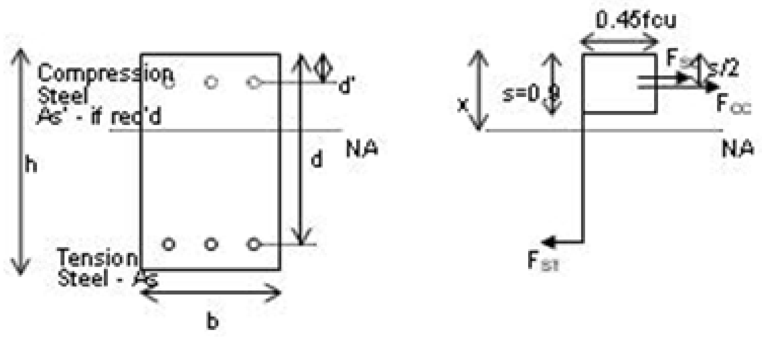
```
Combination= 1
Dir                 Anl: Top    Anl: Bot    Minimum
    N (kN)             131.4       157.6
2   M33 (kN.m)          53.6       -26.2        3.2
3   M22 (kN.m)         -16.2         8.1       -2.0

N-max (kN)            1726.0   > Nd  ...OK...
```

In this example, the design ultimate axial load is determined using Cl. 3.8.4.3

$$N_{max} = (0.4 \times f_{cu} \times b_1 \times b_2) + \left(0.75 \times As_{provided} \times f_y\right) = \quad \mathbf{1726\ kN > N_{\ d}, ok}!$$

Cross check of the above solution

The solution can be cross checked using two basic equations given in standard texts. For example: W.H. Mosley and J.H. Bungey, Reinforced Concrete Design, (MacMillan)



1. Resolving forces vertically

N = $F_{cc} + F_{ST} + F_{sc}$

Where:

- F_{CC} is Concrete Compressive Strength
- F_{ST} is Steel Tensile Force
- F_{SC} is Steel Compressive Force

Bars in tension are fully stressed, hence Total Tensile force in bars at X2 and X3.

$$F_{ST} = -4 \times \frac{As_{req}}{6} \times \frac{460\ {}^{N}\!/_{mm^2}}{1.15} = -164.56\ kN$$

Compressive force in concrete, using the BS8110 rectangular stress.

$$F_{CC} = \frac{0.67 f_{cu}}{1.5} \times (0.9 \times Y - bar \times h) = 244.52\ kN$$

Total compressive force in bars at X1, F_{SC} = N – F_{ST} – F_{CC} = 157.3 – (-164.56) – (244.52) = <u>77.34 kN</u>

2. **Taking moments about mid-depth of section (should equate to zero):**

The applied moment M_y’ must be balanced by the moment of resistance of the forces developed within the cross section.

- Distance to centre of concrete compression force $X_{CC} = \frac{500mm}{2} - 0.9 \times \frac{Y-bar}{2} = 213.5\ mm$
- Distance to centre of steel compression force $X_{SC} = \frac{500mm}{2} - \text{X1} = 214\ mm$
- Distance to centre of steel tension force $X_{ST} = \left(\frac{X3+X2}{2}\right) - \frac{500mm}{2} = 107\ mm$

$$M_y' - (X_{cc} \times F_{cc}) + (X_{ST} \times F_{ST}) - (X_{sc} \times F_{sc}) = 0.31\ kNm$$

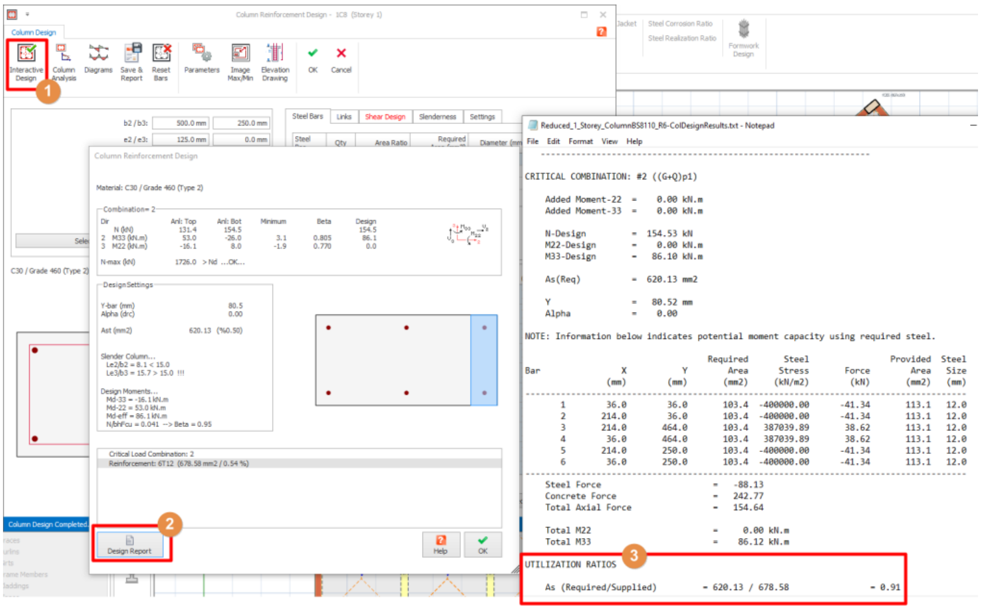
The right hand side of the above equation should equate to zero to within an acceptable tolerance. To determine if this result is OK, recalculate the actual value of M_y’ required for this to be the case and then compare the two.

$$actual\ M_y' = (X_{cc} \times F_{cc}) - (X_{ST} \times F_{ST}) + (X_{sc} \times F_{sc}) + 0.5 = 86.79\ kNm$$

$$M_y' \big/ actual\ M_y' = 86.8\ kNm \big/ 86.79\ kNm = 1,\ \ ok!$$

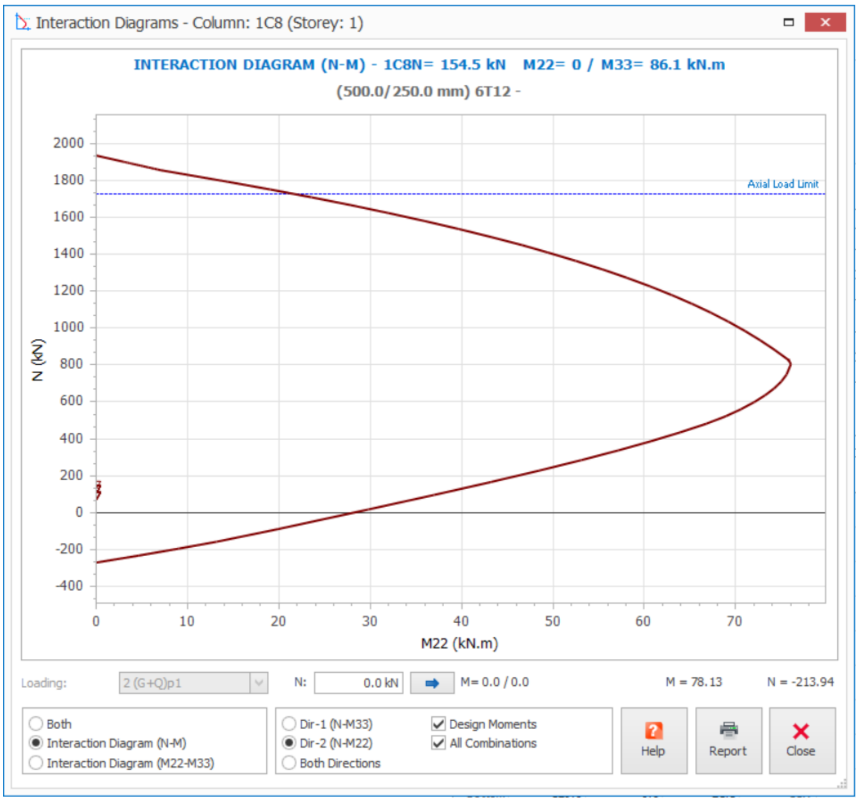
The above cross check shows that if the $As_{required}$ was actually the amount provided then the required capacity is just sufficient.

In all cases As_{prov} will exceed As_{req} to some degree. ProtaStructure reports the ratio: As_{req}/As_{prov} as the utilisation ratio. Utilisation Ratio As_{req} /As_{prov} = 0.91



```
CRITICAL COMBINATION: #2 ((G+Q)p1)

   Added Moment-22  =    0.00 kN.m
   Added Moment-33  =    0.00 kN.m

   N-Design         =  154.53 kN
   M22-Design       =    0.00 kN.m
   M33-Design       =   86.10 kN.m

   As(Req)          =  620.13 mm2

   Y                =   80.52 mm
   Alpha            =    0.00

NOTE: Information below indicates potential moment capacity using required steel.

                                    Required       Steel              Provided  Steel
Bar              X          Y         Area        Stress      Force       Area   Size
               (mm)       (mm)        (mm2)      (kN/m2)       (kN)      (mm2)   (mm)
--------------------------------------------------------------------------------------
       1       36.0       36.0       103.4  -400000.00     -41.34      113.1   12.0
       2      214.0       36.0       103.4  -400000.00     -41.34      113.1   12.0
       3      214.0      464.0       103.4   387039.89      38.62      113.1   12.0
       4       36.0      464.0       103.4   387039.89      38.62      113.1   12.0
       5      214.0      250.0       103.4  -400000.00     -41.34      113.1   12.0
       6       36.0      250.0       103.4  -400000.00     -41.34      113.1   12.0
--------------------------------------------------------------------------------------
   Steel Force                        =   -88.13
   Concrete Force                     =   242.77
   Total Axial Force                  =   154.64

   Total M22                          =     0.00 kN.m
   Total M33                          =    86.12 kN.m

UTILIZATION RATIOS

   As (Required/Supplied)          = 620.13 / 678.58              = 0.91
```

It is important to appreciate that 91% utilisation does not mean that 9% more loads can be added. As is shown on the interaction diagram for this column, a great deal more axial load could be added.



Loading: 2 (G+Q)p1
N: 0.0 kN
M= 0.0 / 0.0
M = 78.13
N = -213.94

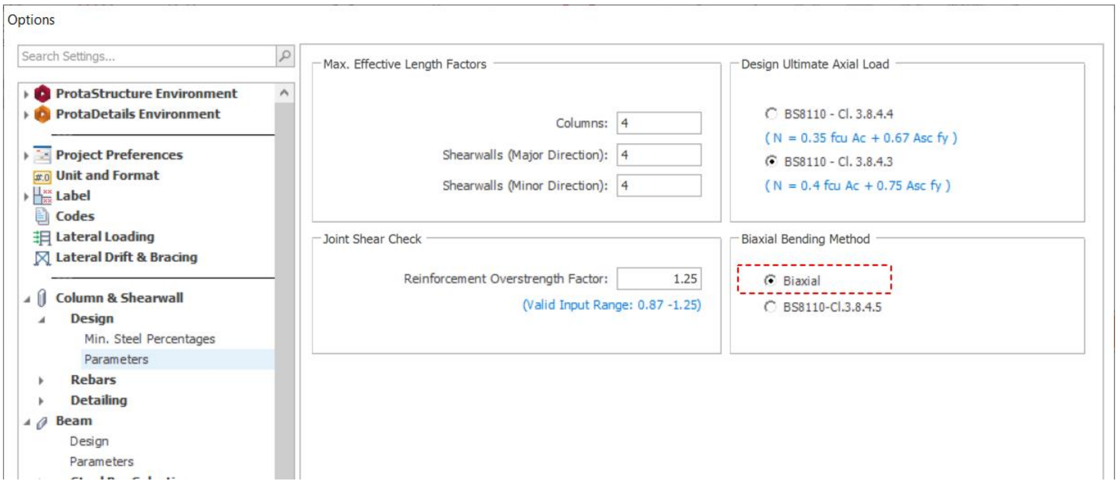
○ Both
● Interaction Diagram (N-M)
○ Interaction Diagram (M22-M33)

○ Dir-1 (N-M33)
● Dir-2 (N-M22)
○ Both Directions

☑ Design Moments
☑ All Combinations

Bi-Axial Design Method Example

From the *Column Design Settings* dialog, change the design method to bi-axial.



Max. Effective Length Factors

Columns: 4

Shearwalls (Major Direction): 4

Shearwalls (Minor Direction): 4

Design Ultimate Axial Load

○ BS8110 - Cl. 3.8.4.4 (N = 0.35 fcu Ac + 0.67 Asc fy)

● BS8110 - Cl. 3.8.4.3 (N = 0.4 fcu Ac + 0.75 Asc fy)

Joint Shear Check

Reinforcement Overstrength Factor: 1.25

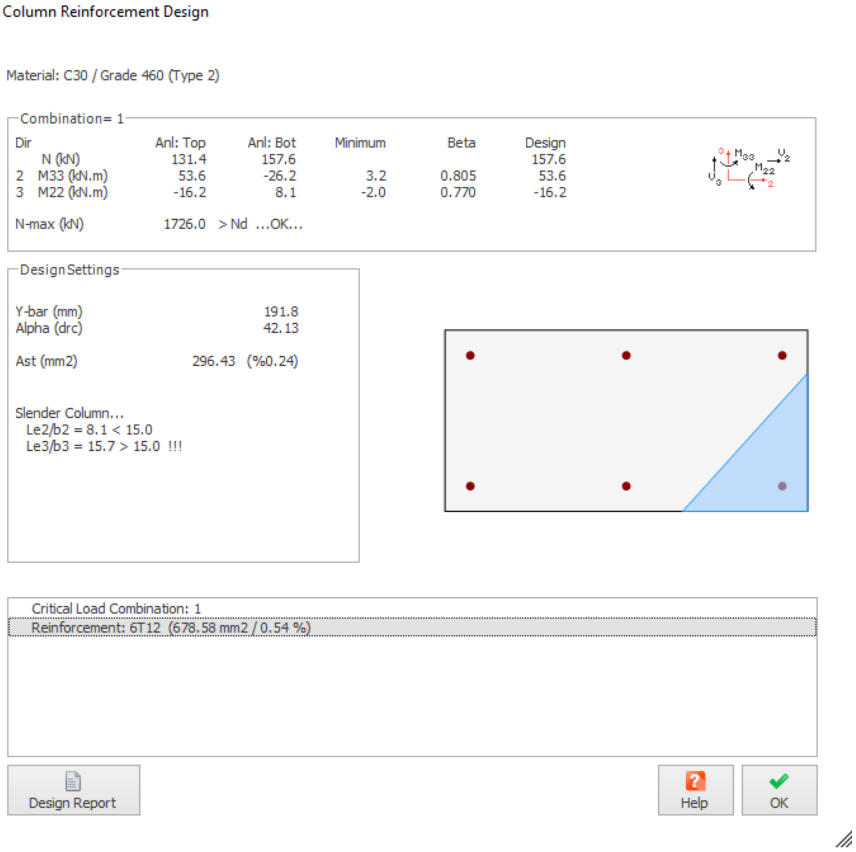
(Valid Input Range: 0.87 -1.25)

Biaxial Bending Method

● Biaxial

○ BS8110-Cl.3.8.4.5

Then re-design column 1C8 once more.



Column Reinforcement Design

Material: C30 / Grade 460 (Type 2)

Combination= 1

```
Dir               Anl: Top   Anl: Bot   Minimum     Beta    Design
    N (kN)          131.4      157.6                         157.6
2   M33 (kN.m)       53.6      -26.2       3.2     0.805      53.6
3   M22 (kN.m)      -16.2        8.1      -2.0     0.770     -16.2

N-max (kN)         1726.0   > Nd  ...OK...
```

Design Settings

```
Y-bar (mm)                    191.8
Alpha (drc)                   42.13

Ast (mm2)              296.43  (%0.24)

Slender Column...
  Le2/b2 = 8.1 < 15.0
  Le3/b3 = 15.7 > 15.0  !!!
```

Critical Load Combination: 1

Reinforcement: 6T12 (678.58 mm2 / 0.54 %)

It is important to note that design stages 1 to 7 are identical to the previous example, hence the effective design moments about each axis are unchanged:

In direction 2 (About direction – 3):

- Smaller end moment, M_1= -26.2 kNm
- Larger end moment, M_2= 53.6 kNm

- $M_i = 0.4\ M_1 + 0.6\ M_2 \geq 0.4\ M_2$ = <u>21.68 kNm ≥ 21.44 kNm</u>
- Hence, $M_{i,2}$ = 21.68 kNm
- $M_{d,33\ eff}$ is the greatest of:
    - a) M_2 =53.6 kNm
    - b) $M_i + M_{add}$ = 21.68 kNm
    - c) $M_1+M_{add}/2$ = 21.68 kNm
    - d) $E_{min}N$ = 3.04 kNm

$M_{d,33\ eff}$ = 53.6 kNm.

<u>In direction 3 (About direction – 2):</u>

- Smaller end moment, M_1= 8.1 kNm
- Larger end moment, M_2= -16.2 kNm
- $M_i = 0.4\ M_1 + 0.6\ M_2 \geq 0.4\ M_2$ = <u>-6.48 kNm ≥ -6.48 kNm</u>
- Hence, M_i = -6.48 kNm
- $M_{d,22\ eff}$ is the greatest of:
    - a) M_2 = -16.2 kNm
    - b) $M_i + M_{add}$ = -1.81 kNm
    - c) $M_1+M_{add}/2$ = 10.435 kNm
    - d) $E_{min}N$ = 1.9 kNm

$M_{d,22\ eff}$ = 16.2 kNm.

Instead of converting these to a uni-axial design moment (as per stage 8), an exact solution is determined using the bi-axial moments.

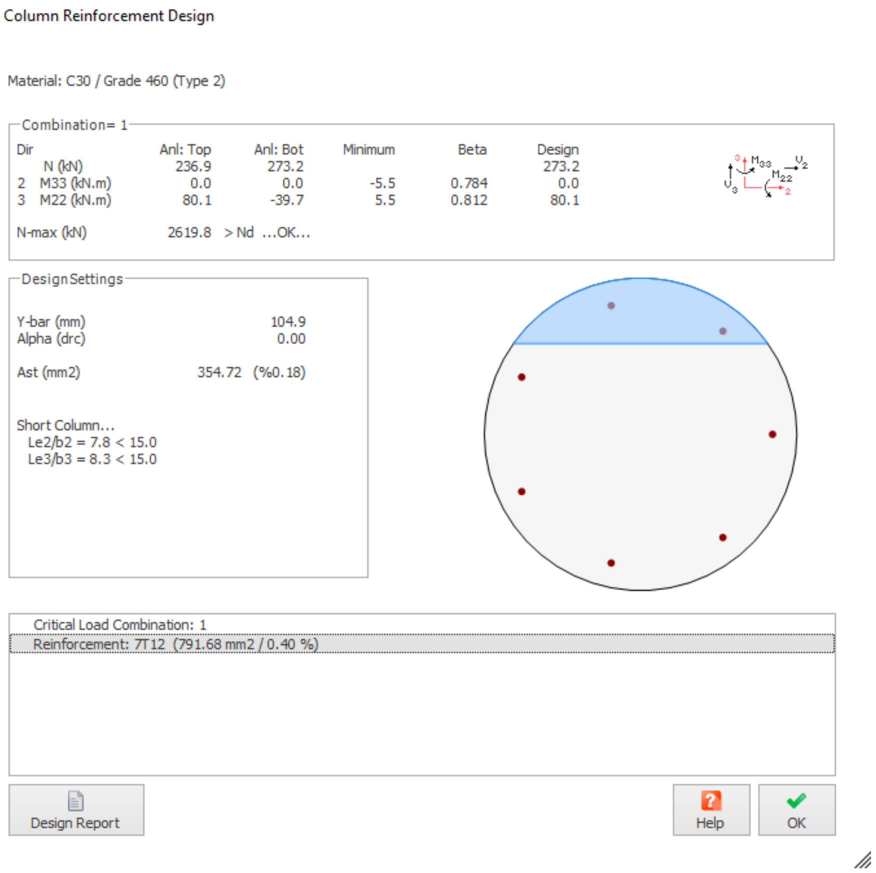
The result is that the area of steel required drops from 636.08 mm^2 to 296.43 mm^2.

Thus, this design method can obviously be seen to provide a more economical solution. The drawback is that because the neutral axis is no longer parallel to either face of the column, verification is more difficult. The cross checks required do not lend themselves to hand calculation.

Braced Circular Column Example

From the Column Design Settings dialog, change the design method to **BS8110-Cl.3.8.4.5.**

Column 1C12 will be used to demonstrate the design process for a circular column.



Column Reinforcement Design

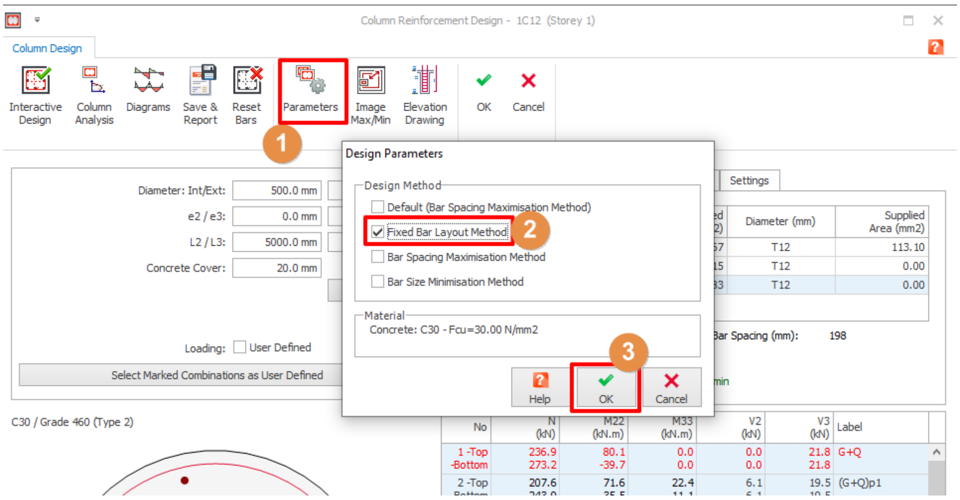
Material: C30 / Grade 460 (Type 2)

```
Combination= 1
Dir              Anl: Top   Anl: Bot   Minimum   Beta    Design
   N (kN)          236.9      273.2                       273.2
2  M33 (kN.m)        0.0        0.0      -5.5    0.784     0.0
3  M22 (kN.m)       80.1      -39.7       5.5    0.812    80.1

N-max (kN)        2619.8  > Nd  ...OK...
```

```
Design Settings

Y-bar (mm)                  104.9
Alpha (drc)                  0.00

Ast (mm2)            354.72  (%0.18)

Short Column...
  Le2/b2 = 7.8 < 15.0
  Le3/b3 = 8.3 < 15.0
```

Critical Load Combination: 1
Reinforcement: 7T12 (791.68 mm2 / 0.40 %)

Click on the Parameters button and change to fixed bar layout.



Column Reinforcement Design - 1C12 (Storey 1)

Diameter: Int/Ext: 500.0 mm
e2 / e3: 0.0 mm
L2 / L3: 5000.0 mm
Concrete Cover: 20.0 mm
Loading: ☐ User Defined
Select Marked Combinations as User Defined

C30 / Grade 460 (Type 2)

Design Parameters

Design Method
☐ Default (Bar Spacing Maximisation Method)
☑ Fixed Bar Layout Method
☐ Bar Spacing Maximisation Method
☐ Bar Size Minimisation Method

Material
Concrete: C30 - Fcu=30.00 N/mm2

Diameter (mm)	Supplied Area (mm2)
T12	113.10
T12	0.00
T12	0.00

Bar Spacing (mm): 198

No	N (kN)	M22 (kN.m)	M33 (kN.m)	V2 (kN)	V3 (kN)	Label
1 -Top -Bottom	236.9 273.2	80.1 -39.7	0.0 0.0	0.0 0.0	21.8 21.8	G+Q
2 -Top	207.6	71.6	22.4	6.1	19.5	(G+Q)p1

This will force the design to adopt the number of bars shown in the 'Qty' cell of the above table. In this example we will use 8 bars in the design, (Qty = 8). When the design is performed, the bar sizes will be adjusted to obtain an economic solution based on this layout.

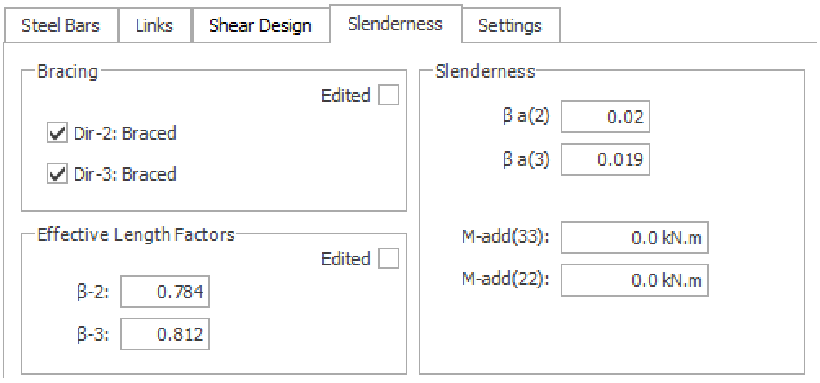
Column Diameter, D = 500 m

The clear height of column in the two directions takes account of the beams framing into the top of the column.

L_{o2} = 5500 mm – 500 mm = **5000 mm**

L_{o3} = 5500 mm – 400 mm = 5**100 mm**

1. Braced or Unbraced – Cl 3.8.1.5



Bracing

Edited ☐

☑ Dir-2: Braced

☑ Dir-3: Braced

Effective Length Factors

Edited ☐

β-2: 0.784

β-3: 0.812

Slenderness

β a(2) 0.02

β a(3) 0.019

M-add(33): 0.0 kN.m

M-add(22): 0.0 kN.m

All the columns in this building should be considered as braced. If the column is designed as such, the effective length factors in the two directions are calculated as:

β_2 = 0.784

β_3 = 0.812

2. Calculate effective height – Cl 3.8.1.6

The values of b_2 and b_2 are calculated in the same way as in the previous rectangular column example. The full calculations will therefore not be repeated here.

The effective height of the column is calculated as:

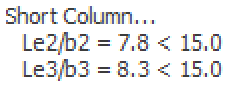
$$l_{e,1} = \beta_2 l_{o2} = 3920\ mm$$

$$l_{e,2} = \beta_3 l_{o3} = 4141\ mm$$

3. Check Slenderness limits – Cl 3.8.1.7 & 3.8.1.8

$$60 \times D = 30000\ mm, \quad 5000\ or\ 5100 < 30000\ mm, ok!$$

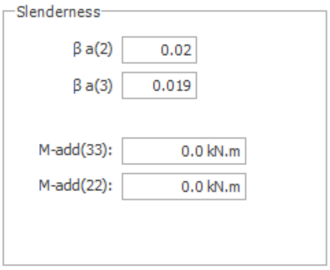
4. Classify as short or slender – Cl 3.8.1.3



```
Short Column...
  Le2/b2 = 7.8 < 15.0
  Le3/b3 = 8.3 < 15.0
```

1C12 was classified as short column.

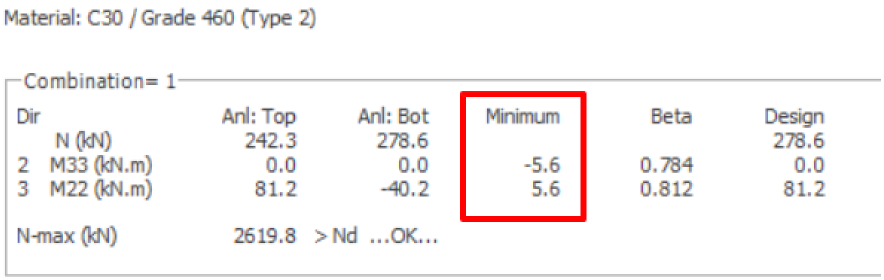
5. If slender – Calculate M_{add} – Cl 3.8.3.1



```
Slenderness
  β a(2)        0.02
  β a(3)        0.019

  M-add(33):    0.0 kN.m
  M-add(22):    0.0 kN.m
```

In this example, the round column was classified as short, hence no additional moments in direction 2 and 3.

6. Calculate minimum moments – Cl 3.8.2.4

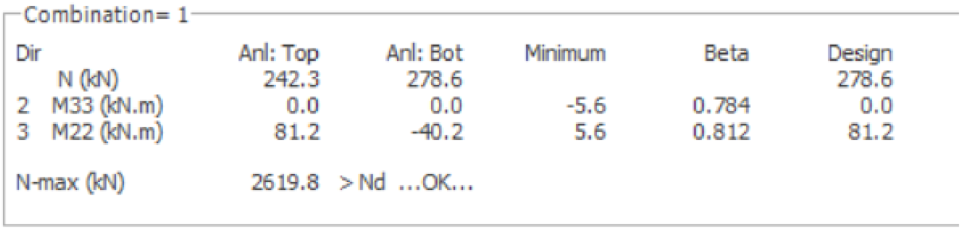


```
Material: C30 / Grade 460 (Type 2)

Combination= 1
Dir                 Anl: Top   Anl: Bot   Minimum    Beta    Design
    N (kN)             242.3      278.6                       278.6
2   M33 (kN.m)           0.0        0.0      -5.6    0.784      0.0
3   M22 (kN.m)          81.2      -40.2       5.6    0.812     81.2

N-max (kN)            2619.8  > Nd  ...OK...
```

- Applied Axial Load, N= **278.6 kN**
- Minimum eccentricity = min (0.05 x d, 20 mm) = 20.0 mm
- M_{min} = N x 20 mm= **5.57 kNm**

When the column was braced, combination 1 was identified as the critical combination.

7. Calculate design moments about each axis – Cl 3.8.3.2



```
Combination= 1
Dir                 Anl: Top   Anl: Bot   Minimum    Beta    Design
    N (kN)             242.3      278.6                       278.6
2   M33 (kN.m)           0.0        0.0      -5.6    0.784      0.0
3   M22 (kN.m)          81.2      -40.2       5.6    0.812     81.2

N-max (kN)            2619.8  > Nd  ...OK...
```

<u>In direction 3 (About direction-2):</u>

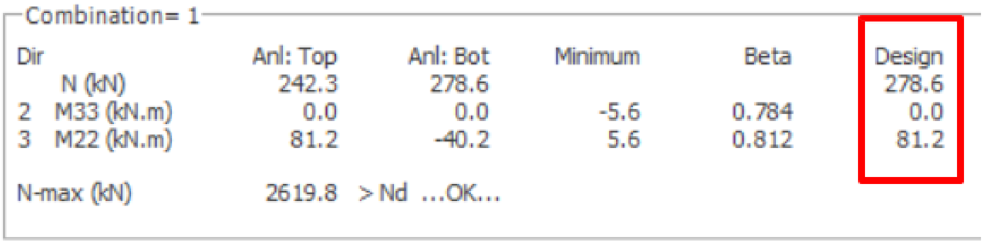
- Smaller end moment, M_1 = -40.2 kNm
- Larger end moment, M_2= 81.2 kNm

As the column is short the effective moment is simply greatest of:

- M_2 = 81.2 kNm
- M_{min}= **5.6 kNm**

$M_{d,33\ eff}$ = 81.2 kNm.

8. Calculate equivalent uni-axial design moments – Cl 3.8.4.5



```
Combination= 1
Dir                 Anl: Top    Anl: Bot    Minimum      Beta      Design
    N (kN)             242.3       278.6                              278.6
2   M33 (kN.m)           0.0         0.0       -5.6      0.784          0.0
3   M22 (kN.m)          81.2       -40.2        5.6      0.812         81.2

N-max (kN)            2619.8   > Nd  ...OK...
```

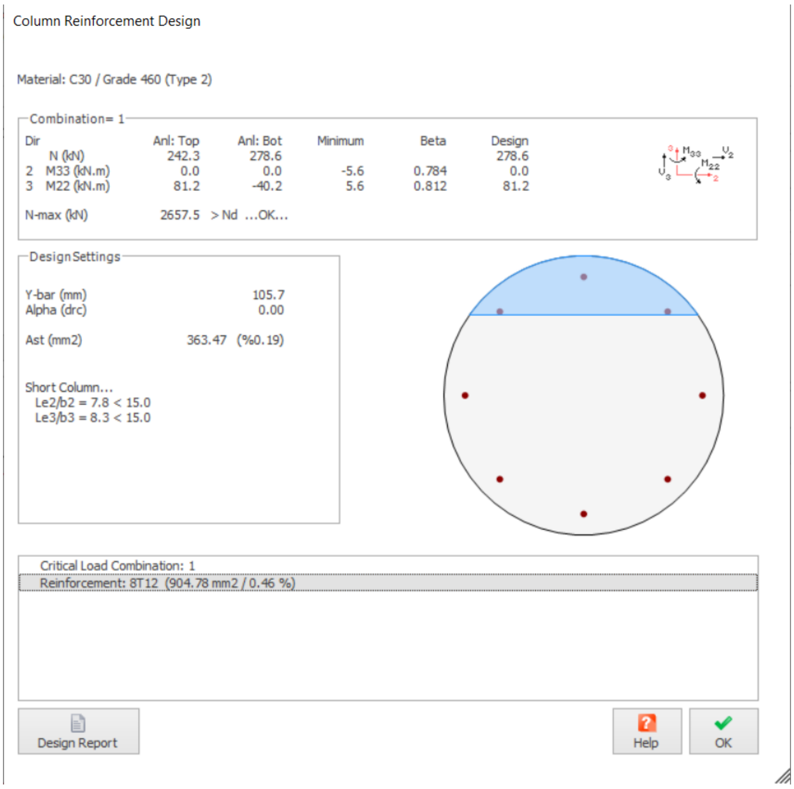
$M_x = M_{d,33\ eff} = 0$ kNm.

$M_y = M_{d,22\ eff} = 81.2$ kNm.

$$M'_x = \sqrt{(M_x)^2 + \left(M_y\right)^2} = 81.2\ kNm$$

9. Member Design – Cl 3.8.4

Solution determined by ProtaStructure:



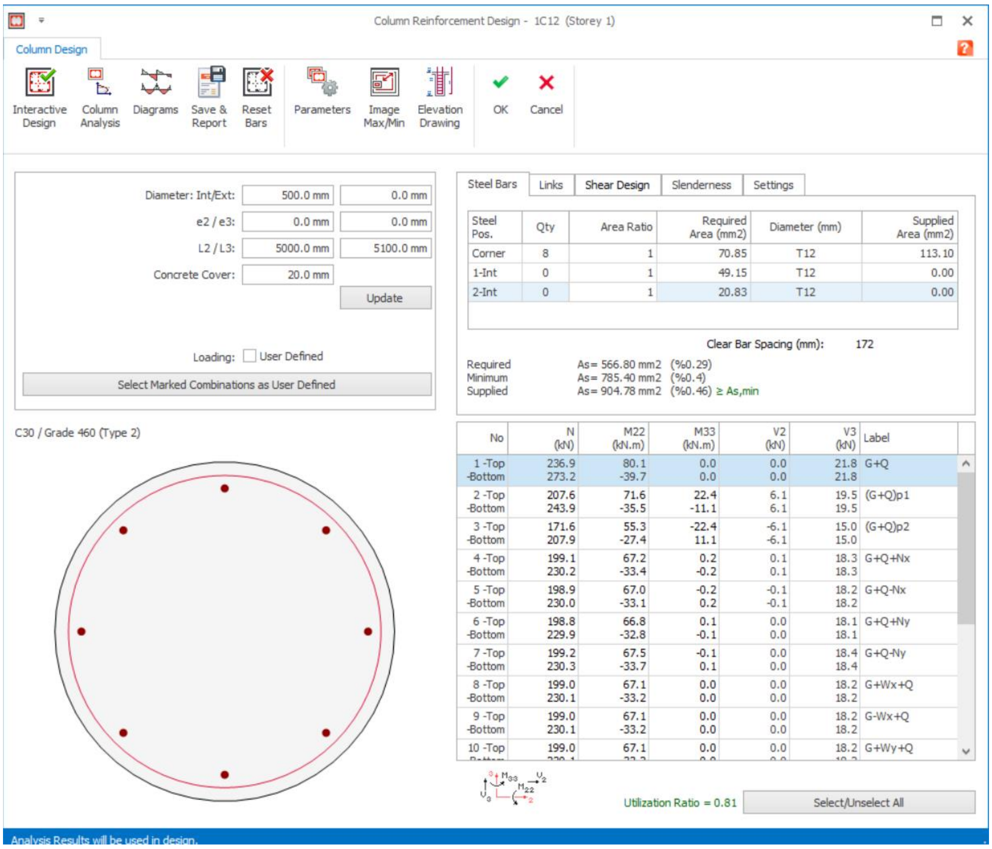
```
Column Reinforcement Design

Material: C30 / Grade 460 (Type 2)

Combination= 1
Dir                 Anl: Top    Anl: Bot    Minimum      Beta      Design
    N (kN)             242.3       278.6                              278.6
2   M33 (kN.m)           0.0         0.0       -5.6      0.784          0.0
3   M22 (kN.m)          81.2       -40.2        5.6      0.812         81.2

N-max (kN)            2657.5   > Nd  ...OK...

Design Settings

Y-bar (mm)                   105.7
Alpha (drc)                   0.00

Ast (mm2)           363.47  (%0.19)

Short Column...
  Le2/b2 = 7.8 < 15.0
  Le3/b3 = 8.3 < 15.0

Critical Load Combination: 1
Reinforcement: 8T12  (904.78 mm2 / 0.46 %)
```

- Distance to neutral axis-Y bar = 105.7 mm
- Area of steel required, $As_{required} = 363.47\ mm^2$
- Area of steel provided, $As_{provided} = 904.78\ mm^2$
- No. of provided reinforcement = 8 bars

Before cross checking this solution for equilibrium, we will first make the column unbraced.

Unbraced Circular Column Example

The previous calculations are now repeated with the column specified as unbraced.



Diameter: Int/Ext: 500.0 mm 0.0 mm
e2 / e3: 0.0 mm 0.0 mm
L2 / L3: 5000.0 mm 5100.0 mm
Concrete Cover: 20.0 mm
Loading: ☐ User Defined

C30 / Grade 460 (Type 2)

Steel Pos.	Qty	Area Ratio	Required Area (mm2)	Diameter (mm)	Supplied Area (mm2)
Corner	8	1	70.85	T12	113.10
1-Int	0	1	49.15	T12	0.00
2-Int	0	1	20.83	T12	0.00

Clear Bar Spacing (mm): 172

Required As= 566.80 mm2 (%0.29)
Minimum As= 785.40 mm2 (%0.4)
Supplied As= 904.78 mm2 (%0.46) ≥ As,min

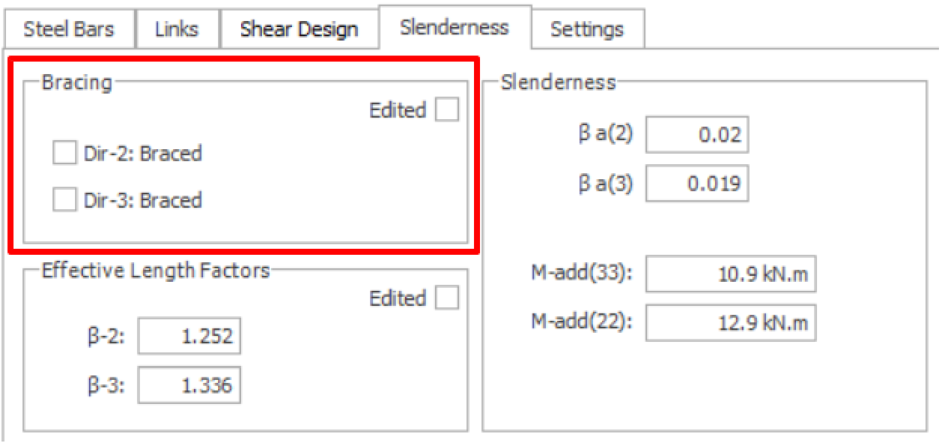
No	N (kN)	M22 (kN.m)	M33 (kN.m)	V2 (kN)	V3 (kN)	Label
1 -Top -Bottom	236.9 273.2	80.1 -39.7	0.0 0.0	0.0 0.0	21.8 21.8	G+Q
2 -Top -Bottom	207.6 243.9	71.6 -35.5	22.4 -11.1	6.1 6.1	19.5 19.5	(G+Q)p1
3 -Top -Bottom	171.6 207.9	55.3 -27.4	-22.4 11.1	-6.1 -6.1	15.0 15.0	(G+Q)p2
4 -Top -Bottom	199.1 230.2	67.2 -33.4	0.2 -0.2	0.1 0.1	18.3 18.3	G+Q+Nx
5 -Top -Bottom	198.9 230.0	67.0 -33.1	-0.2 0.2	-0.1 -0.1	18.2 18.2	G+Q-Nx
6 -Top -Bottom	198.8 229.9	66.8 -32.8	0.1 -0.1	0.0 0.0	18.1 18.1	G+Q+Ny
7 -Top -Bottom	199.2 230.3	67.5 -33.7	-0.1 0.1	0.0 0.0	18.4 18.4	G+Q-Ny
8 -Top -Bottom	199.0 230.1	67.1 -33.2	0.0 0.0	0.0 0.0	18.2 18.2	G+Wx+Q
9 -Top -Bottom	199.0 230.1	67.1 -33.2	0.0 0.0	0.0 0.0	18.2 18.2	G-Wx+Q
10 -Top	199.0	67.1	0.0	0.0	18.2	G+Wy+Q

Utilization Ratio = 0.81

Analysis Results will be used in design.

1. Braced or Unbraced – Cl 3.8.1.5

To change the column to unbraced, check 'Edited' box and uncheck the 'Dir 2' and 'Dir 3' braced boxes on the slenderness tab as shown below. Then, click the Design button:



Bracing: Edited ☐
☐ Dir-2: Braced
☐ Dir-3: Braced

Effective Length Factors: Edited ☐
β-2: 1.252
β-3: 1.336

Slenderness:
β a(2) 0.02
β a(3) 0.019
M-add(33): 10.9 kN.m
M-add(22): 12.9 kN.m

When unbraced, the effective length factors in the two directions for this column change to:

$\beta_2 = 1.252$

$\beta_3 = 1.336$

2. Calculate effective height – Cl 3.8.1.6

When considered unbraced, the column effective height is calculated as:

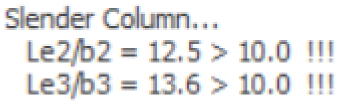
$$l_{e,2} = \beta_2 l_{o2} = 6260\ mm$$

$$l_{e,3} = \beta_3 l_{o3} = 6810\ mm$$

3. Check Slenderness limits – Cl 3.8.1.7 & 3.8.1.8

$$60 \times D = 30000\ mm, \quad 5000\ and\ 5100 < 30000\ mm, ok!$$

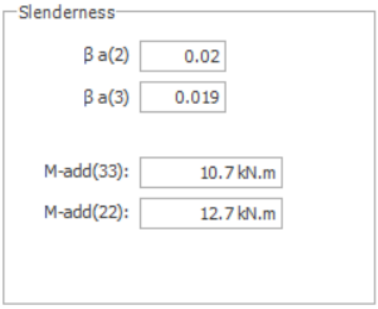
4. Classify as short or slender – Cl 3.8.1.3



```
Slender Column...
  Le2/b2 = 12.5 > 10.0  !!!
  Le3/b3 = 13.6 > 10.0  !!!
```

When 1C12 is unbraced, it is classified as slender column.

5. If slender – Calculate M_{add} – Cl 3.8.3.1

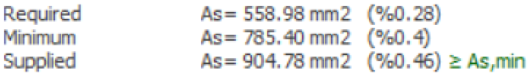


β a(2) 0.02

β a(3) 0.019

M-add(33): 10.7 kN.m

M-add(22): 12.7 kN.m



```
Required     As= 558.98 mm2  (%0.28)
Minimum      As= 785.40 mm2  (%0.4)
Supplied     As= 904.78 mm2  (%0.46) ≥ As,min
```

<u>In Direction 2</u>

$$\beta_{a(2)} = \frac{1}{2000}\left(\frac{l_e}{b'}\right)^2 = 0.078 \qquad Equation\ 34$$

<u>In Direction 3</u>

$$\beta_{a(3)} = \frac{1}{2000}\left(\frac{l_e}{b'}\right)^2 = 0.093 \qquad Equation\ 34$$

Column is slender in direction 2 and 3, hence $M_{add,1}$ and $M_{add,2}$ must be calculated:

$$K = \frac{N_{uz} - N}{N_{uz} - N_{bal}} \leq 1$$

- Applied Axial Load, N= <u>278.6 kN</u>

- Area of the column section, A=196349.54 mm^2
- Concrete Grade, f_{cu}=30 N/mm^2
- Steel Grade, f_y= 460 N/mm^2
- Material factor for steel, s= 1.150
- Area of steel required, As_{req}= 568.49 mm^2

$$N_{uz} = 0.45 f_{cu} A_c + 0.87 f_y A_{sc}$$
$$= 0.45 \times 30 \times \left[(A) - As_{req}\right] + 0.87 \times f_y \times As_{req} = \mathbf{2870.54}\ \boldsymbol{kN}$$

$$N_{bal} = 0.25 f_{cu} A = \mathbf{1472.62}\ \boldsymbol{kN}.$$

$$K = \frac{N_{uz} - N}{N_{uz} - N_{bal}} = 1.85, \quad \textbf{hence K = 1}$$

$$\alpha_{u1} = \beta_{a(2)} KD = \mathbf{39}\ \boldsymbol{mm} \qquad Equation\ 32$$

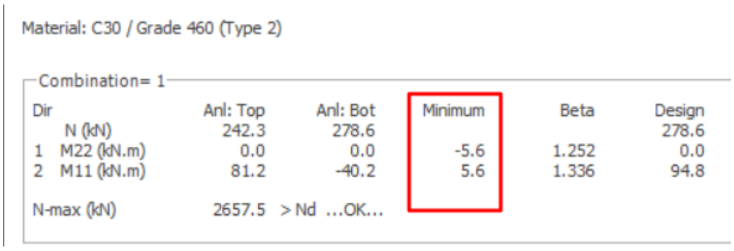
$$M_{add,33} = N\alpha_{u1} = \mathbf{10.87}\ \boldsymbol{kNm} \qquad Equation\ 35$$

$$\alpha_{u2} = \beta_{a(3)} KD = \mathbf{46.5}\ \boldsymbol{mm} \qquad Equation\ 32$$

$$M_{add,22} = N\alpha_{u2} = \mathbf{12.95}\ \boldsymbol{kNm} \qquad Equation\ 35$$

Hence, additional moment about direction - 3 is **10.87 kNm** and direction -2 is **12.95kNm.**

6. Calculate minimum moments – Cl 3.8.2.4



Material: C30 / Grade 460 (Type 2)

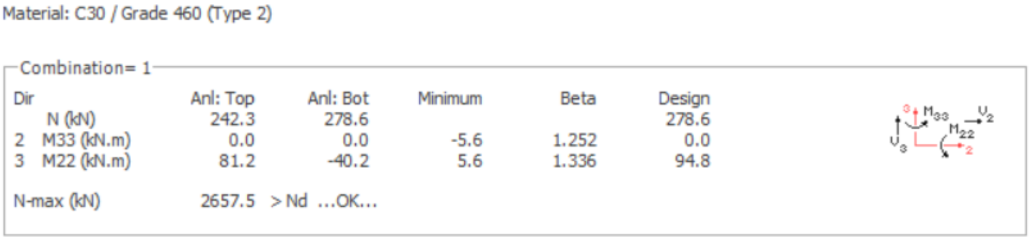
Combination= 1

Dir		Anl: Top	Anl: Bot	Minimum	Beta	Design
	N (kN)	242.3	278.6			278.6
1	M22 (kN.m)	0.0	0.0	-5.6	1.252	0.0
2	M11 (kN.m)	81.2	-40.2	5.6	1.336	94.8
	N-max (kN)	2657.5	> Nd ...OK...			

Minimum eccentricity = min (0.05 x D, 20 mm) = 20.0 mm

M_{min}=N x 20 mm= **5.57 kNm**

7. Calculate unbraced design moments about each axis – Cl 3.8.3.2



Material: C30 / Grade 460 (Type 2)

Combination= 1

Dir		Anl: Top	Anl: Bot	Minimum	Beta	Design
	N (kN)	242.3	278.6			278.6
2	M33 (kN.m)	0.0	0.0	-5.6	1.252	0.0
3	M22 (kN.m)	81.2	-40.2	5.6	1.336	94.8
	N-max (kN)	2657.5	> Nd ...OK...			

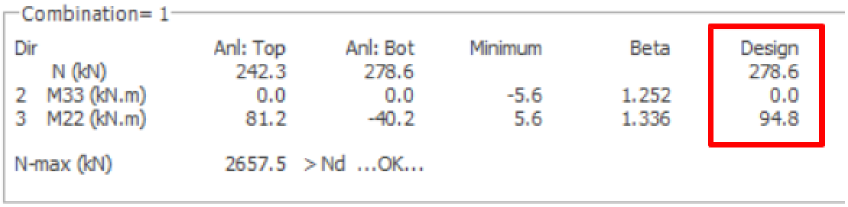
<u>In direction 3 (About direction – 2):</u>

- Smaller end moment, M_1= -40.2 kNm
- Larger end moment, M_2= 81.2 kNm
- $M_{d,22\ eff}$ is the greatest of:

a) $M_2 + M_{add,22} = 94.15$ kNm
b) $M_1 + M_{add,22} = -27.25$ kNm
c) $E_{min}N = 5.57$ kNm

$M_{d,22\ eff} = 94.15$ kNm.

8. Calculate equivalent uni-axial design moments – Cl 3.8.4.5



```
Combination= 1
Dir                 Anl: Top    Anl: Bot    Minimum      Beta      Design
    N (kN)             242.3       278.6                              278.6
2   M33 (kN.m)           0.0         0.0       -5.6     1.252          0.0
3   M22 (kN.m)          81.2       -40.2        5.6     1.336         94.8

N-max (kN)            2657.5   > Nd  ...OK...
```

$M_x = M_{d,22\ eff} = 94.8$ kNm

$M_y = M_{d,33\ eff} = 0$ kNm

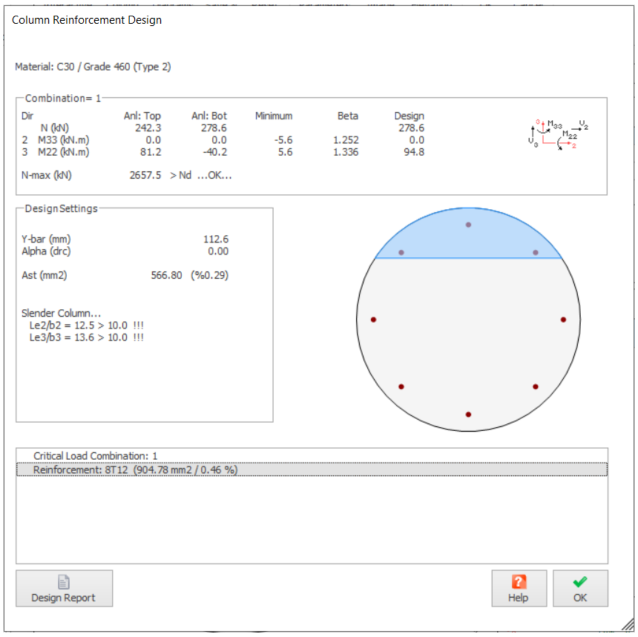
$$M = \sqrt{\left({M_x}^2 + {M_y}^2\right)} = \mathbf{94.8\ kNm}$$

9. Member Design – Cl 3.8.4

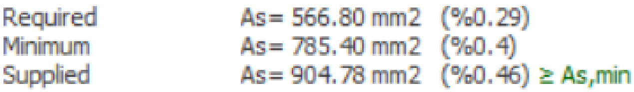
Design forces:

- N = 278.6 kN
- M = 94.8 kNm

Solution determined by ProtaStructure:



```
Column Reinforcement Design

Material: C30 / Grade 460 (Type 2)

Combination= 1
Dir                 Anl: Top    Anl: Bot    Minimum      Beta      Design
    N (kN)             242.3       278.6                              278.6
2   M33 (kN.m)           0.0         0.0       -5.6     1.252          0.0
3   M22 (kN.m)          81.2       -40.2        5.6     1.336         94.8

N-max (kN)            2657.5   > Nd  ...OK...

Design Settings
Y-bar (mm)                        112.6
Alpha (drc)                        0.00

Ast (mm2)                566.80  (%0.29)

Slender Column...
  Le2/b2 = 12.5 > 10.0  !!!
  Le3/b3 = 13.6 > 10.0  !!!

Critical Load Combination: 1
Reinforcement: 8T12  (904.78 mm2 / 0.46 %)
```

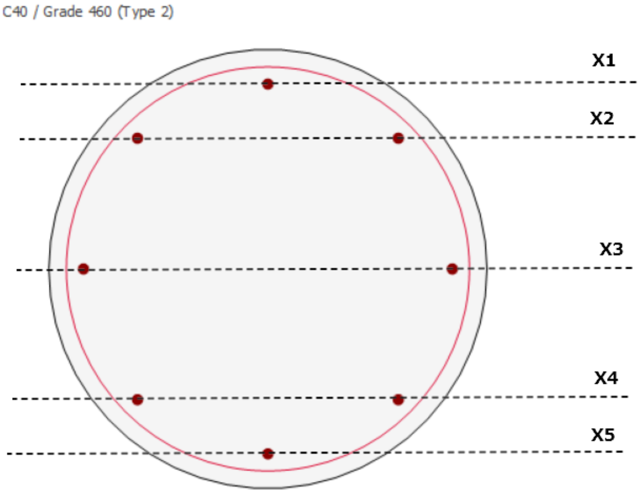


Required	As= 566.80 mm2 (%0.29)
Minimum	As= 785.40 mm2 (%0.4)
Supplied	As= 904.78 mm2 (%0.46) ≥ As,min

- Distance to neutral axis Y-bar = 112.6 mm
- Area of steel required, $As_{required}$ = 566.80 mm^2
- Area of steel provided, $As_{provided}$ = 904.78 mm^2
- No. of reinforcement provided = 8 bars

Cross check of the above solution

Reinforcement in the section:



- Diameter, D = 500 mm
- Bar diameter, d = 12 mm
- Cover = 20 mm
- Links = 10 mm

<u>Bar distances from mid depth:</u>

- X1 = D/2 – cover – links – bar diameter/2 = 214 mm
- X2 = $\sqrt{\left(\frac{X1^2}{2}\right)} = 151.32\ mm$
- X3 = 0
- X4 = X2 = 151.32 mm
- X5 = X1 = 214 mm

Y-bar =112.8 > (250 mm – X2 = 98.68 mm), hence bars at X1 and X2 are in compression

1. Resolving forces vertically

N = F_{cc} + F_{ST} + F_{sc}

Where:

- F_{CC} is Concrete Compressive Strength

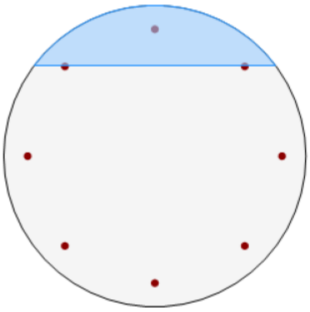
- F_{ST} is Steel Tensile Force
- F_{SC} is Steel Compressive Force

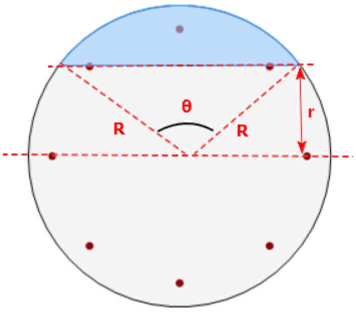
Bars in tension are fully stressed, hence Total Tensile force in bars at X3, X4 and X5.

$$F_{ST} = -5 \times \frac{As_{req}}{nBar} \times \frac{460\ N/_{mm^2}}{1.15} = -142.12\ kN$$

Tensile force per bar at X3, X4 and X5

$$F_{STbar} = \frac{F_{ST}}{5} = -28.42\ kN$$





The area of concrete in compression (the blue shaded area above) is determined from the equation:

$$R = {}^{D}/_{2} = 250\ mm$$

$$r = R - Ybar = 137.2\ mm$$

$$\cos\frac{\theta}{2} = \frac{r}{R}$$

$$\theta = \mathbf{113.27}°$$

$$A = \frac{R^2}{2}\left(\frac{\pi}{180}\theta - \sin\theta\right) = \mathbf{33071.28\ mm^2}$$

Compressive force in concrete, using the BS8110 rectangular stress:

$$F_{CC} = \frac{0.67 f_{cu}}{1.5} \times (0.9 \times A) = 398.84\ kN$$

Total compressive force in bars at X1 and X2:

F_{SC}= N – F_{ST} – F_{CC} = 278.6 – (-142.12) – (398.84) = 21.88 kN

Compressive force per bar at X1 and X2:

F_{SCbar} = F_{SC} /3= 7.29 kN

2. Taking moments about mid-depth of section (should equate to zero):

For this hand calculation it has been assumed that the centre of concrete compression force is at 2Y-bar/3 from the top of the section. The software would of course perform a rigorous calculation to determine the exact position of the centre of concrete compression force. Distance to centre of concrete compression force.

- Distance to centre of concrete compression force X$_{CC} = R - \frac{2\times Ybar}{3} = \ 174.8\ mm$

$$Actual\ M_y' = (X_{cc} \times F_{cc}) - (2 \times X4 \times F_{STbar}) - (X5 \times F_{STbar}) + (X1 \times F_{SCbar}) + (2 \times X2 \times F_{SCbar}) = \mathbf{88.17\ kNm}$$

$$M_y' \Big/ actual\ M_y' = 94.8\ kNm \big/ 88.17\ kNm = \mathbf{1.08}, \quad ok!$$

$$Utilisatio\ Ratio = As_{req} \Big/ As_{prov} = \mathbf{0.63}, \quad ok!$$

Interaction Diagrams - Column: 1C12 (Storey: 1)

INTERACTION DIAGRAM (N-M) - 1C12N= 273 kN M22= 93.4 / M33= 0 kN.m

(D=500mm) 7T12 -

Loading: 1 G+Q

N: 0.0 kN

M= 0.0 / 0.0

M = 24.90 N = 3,253.63

○ Both
● Interaction Diagram (N-M)
○ Interaction Diagram (M22-M33)

○ Dir-1 (N-M33)
● Dir-2 (N-M22)
○ Both Directions

☑ Design Moments
☑ All Combinations

Thank You...

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