REFERENCES	CALCULATIONS	RESULTS

MEMBER #1 (SECTION POSITION 2500.0 mm) BEAM DESIGN REPORT

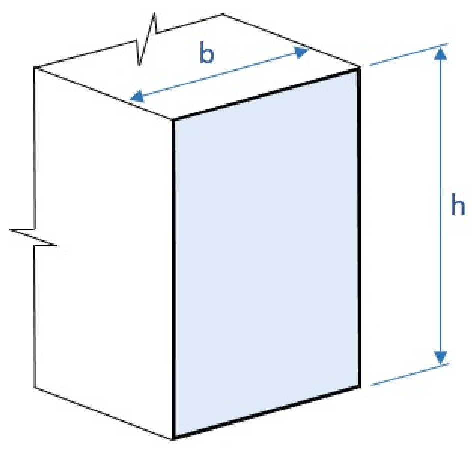
Code: AS 3600-2009

Project details

Project Name:
Project ID:
Company:
Designer:
Client:
Project Notes:
Project Units: Metric

General member design information

Dimentions:



Height $h = 550\ \mathrm{mm}$
Width $b = 250\ \mathrm{mm}$
Member length = $5000\ \mathrm{mm}$

Material properties:
Concrete strength $\acute{f_c} = 50\ \mathrm{MPa}$
Steel strength of longitudinal rebar $f_{sy} = 500\ \mathrm{MPa}$
Steel strength of shear rebar $f_{syv} = 500\ \mathrm{MPa}$

Design Factors and Settings:
Reinforcement Class : N

Load Combinations (Ultimate Limit State)

For axial force in section:
LC1: USER = $0\ \mathrm{kN}$

For bending moment in section:
LC1: USER = $278.82\ \mathrm{kN\text{-}m}$

For shear force in section:
LC1: USER = $0\ \mathrm{kN}$

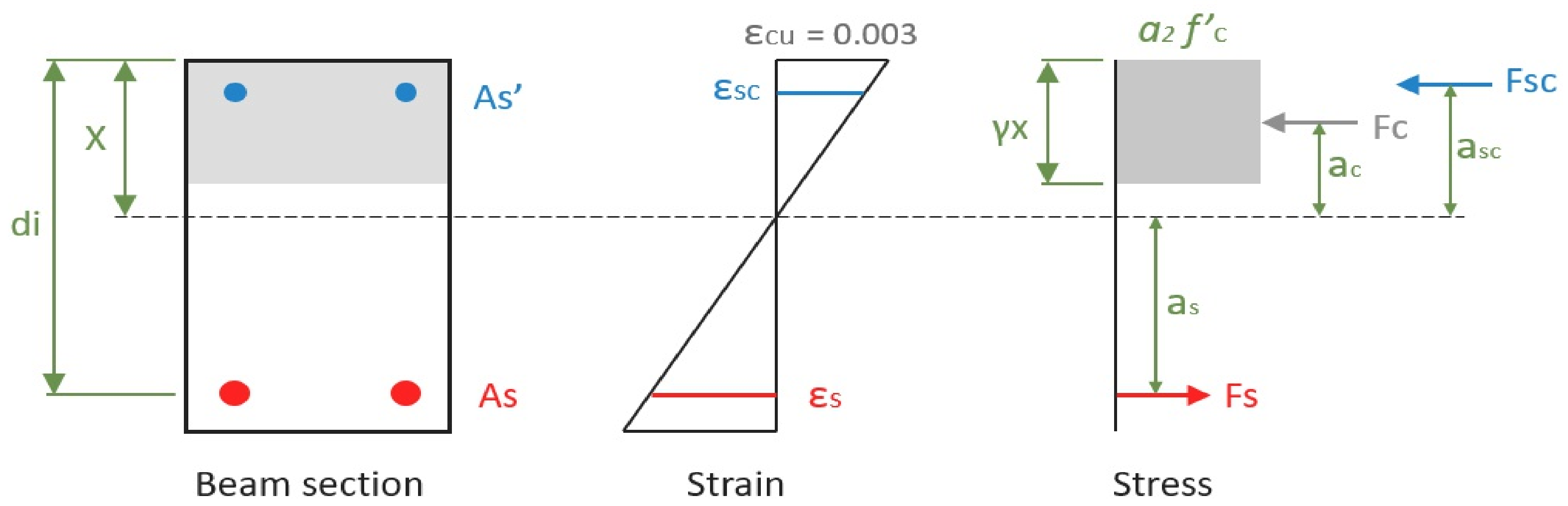
Load Combinations (Serviceability Limit State)

For bending moment in section:
LC1: USER = $150\ \mathrm{kN\text{-}m}$

8.1.3(1), 8.1.3(2), Table 2.2.2, 8.1.5, 2.2.2, 8.1.6.1(2)

Flexure check (Positive bending moment case)

BENDING MOMENT CAPACITY



Section input data:
Design yield strain of rebar $e_y = f_{sy}/E_s = 500/200000 = 0.00250$
Ultimate strain in concrete $e_{cu} = 0.003$
Distance to the outermost layer of tensile reinforcement $d_0 = 500\ \mathrm{mm}$
Given bending moment $M^* = 278.82\ \mathrm{kN\text{-}m}$

Section Rebar

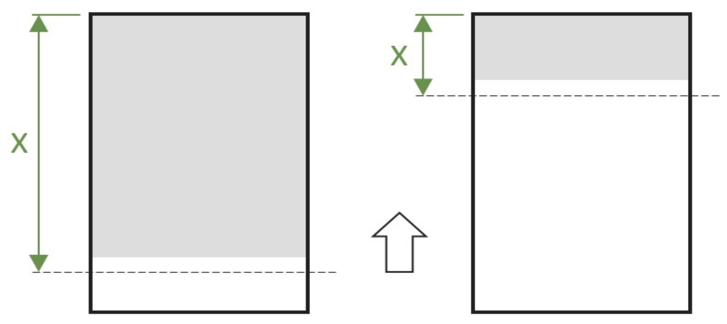
Depth di (mm)	bar diameter (mm)	bar area Asi (mm2)
500	30.902	750.00
500	30.902	750.00

Rectangular compression block factors (8.1.3(1), 8.1.3(2))

$$\alpha_2 = 1.0 - 0.003 \cdot \acute{f_c} = 1.0 - 0.003 \cdot 50 = 0.85$$

$$\gamma = 1.05 - 0.007 \cdot \acute{f_c} = 1.05 - 0.007 \cdot 50 = 0.70$$

1. Calculation of neutral axis depth x



Calculation is based on iterative process:
- Assume x
- Calculate concrete force $F_c = \alpha_2 \cdot \acute{f_c} \cdot \int_{dA} \cdot \gamma \cdot x$
- Calculate compression force in steel $F_{cs} = \sum \acute{A_{s,i}} \cdot f_{s,i}$
- Calculate tensioning force in steel $F_s = \sum A_{s,i} \cdot f_{s,i}$
- Check equilibrium $F_c + F_{cs} = F_s$

Reinforcement stresses $f_s = \{e_s E_s \ (e_s \leq e_y), \ e_y \ (e_s > e_y)\}$
Reinforcement strains above axis $e_s = e_{cu} \cdot (x - d)/x$
Reinforcement strains below axis $e_s = e_{cu} \cdot (d - x)/x$

Searching of neutral axis x (from 500 to 0 mm)

Iter.	x (mm)	kuo = x/do	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	500.0	1.00	3718.75	0.00	3718.75	0.00	Infinity
2	490.0	0.98	3644.38	0.00	3644.38	18.37	198.416
3	480.0	0.96	3570.00	0.00	3570.00	37.50	95.200
4	470.0	0.94	3495.63	0.00	3495.63	57.45	60.850
5	460.0	0.92	3421.25	0.00	3421.25	78.26	43.716
6	450.0	0.90	3346.88	0.00	3346.88	100.00	33.469
7	440.0	0.88	3272.50	0.00	3272.50	122.73	26.665
8	430.0	0.86	3198.13	0.00	3198.13	146.51	21.828
9	420.0	0.84	3123.75	0.00	3123.75	171.43	18.222
10	410.0	0.82	3049.38	0.00	3049.38	197.56	15.435
11	400.0	0.80	2975.00	0.00	2975.00	225.00	13.222
12	390.0	0.78	2900.63	0.00	2900.63	253.85	11.427
13	380.0	0.76	2826.25	0.00	2826.25	284.21	9.944
14	370.0	0.74	2751.88	0.00	2751.88	316.22	8.703
15	360.0	0.72	2677.50	0.00	2677.50	350.00	7.650
16	350.0	0.70	2603.12	0.00	2603.12	385.71	6.749
17	340.0	0.68	2528.75	0.00	2528.75	423.53	5.971
18	330.0	0.66	2454.37	0.00	2454.37	463.64	5.294
19	320.0	0.64	2380.00	0.00	2380.00	506.25	4.701
20	310.0	0.62	2305.63	0.00	2305.63	551.61	4.180
21	300.0	0.60	2231.25	0.00	2231.25	600.00	3.719
22	290.0	0.58	2156.88	0.00	2156.88	651.72	3.309

23	280.0	0.56	2082.50	0.00	2082.50	707.14	2.945
24	270.0	0.54	2008.13	0.00	2008.13	750.00	2.678
25	260.0	0.52	1933.75	0.00	1933.75	750.00	2.578
26	250.0	0.50	1859.38	0.00	1859.38	750.00	2.479
27	240.0	0.48	1785.00	0.00	1785.00	750.00	2.380
28	230.0	0.46	1710.63	0.00	1710.63	750.00	2.281
29	220.0	0.44	1636.25	0.00	1636.25	750.00	2.182
30	210.0	0.42	1561.88	0.00	1561.88	750.00	2.083
31	200.0	0.40	1487.50	0.00	1487.50	750.00	1.983
32	190.0	0.38	1413.13	0.00	1413.13	750.00	1.884
33	180.0	0.36	1338.75	0.00	1338.75	750.00	1.785
34	170.0	0.34	1264.37	0.00	1264.37	750.00	1.686
35	160.0	0.32	1190.00	0.00	1190.00	750.00	1.587
36	150.0	0.30	1115.63	0.00	1115.63	750.00	1.488
37	140.0	0.28	1041.25	0.00	1041.25	750.00	1.388
38	130.0	0.26	966.88	0.00	966.88	750.00	1.289
39	120.0	0.24	892.50	0.00	892.50	750.00	1.190
40	110.0	0.22	818.13	0.00	818.13	750.00	1.091
(Fc + Fcs) < Fs. Updating of iterations							
1	100.0	0.20	743.75	0.00	743.75	750.00	0.992
2	109.8	0.22	816.64	0.00	816.64	750.00	1.089
3	109.6	0.22	815.15	0.00	815.15	750.00	1.087
4	109.4	0.22	813.66	0.00	813.66	750.00	1.085
5	109.2	0.22	812.17	0.00	812.17	750.00	1.083
6	109.0	0.22	810.69	0.00	810.69	750.00	1.081
7	108.8	0.22	809.20	0.00	809.20	750.00	1.079
8	108.6	0.22	807.71	0.00	807.71	750.00	1.077
9	108.4	0.22	806.22	0.00	806.22	750.00	1.075
10	108.2	0.22	804.74	0.00	804.74	750.00	1.073
11	108.0	0.22	803.25	0.00	803.25	750.00	1.071
12	107.8	0.22	801.76	0.00	801.76	750.00	1.069
13	107.6	0.22	800.27	0.00	800.27	750.00	1.067
14	107.4	0.21	798.79	0.00	798.79	750.00	1.065
15	107.2	0.21	797.30	0.00	797.30	750.00	1.063
16	107.0	0.21	795.81	0.00	795.81	750.00	1.061
17	106.8	0.21	794.32	0.00	794.32	750.00	1.059
18	106.6	0.21	792.84	0.00	792.84	750.00	1.057
19	106.4	0.21	791.35	0.00	791.35	750.00	1.055
20	106.2	0.21	789.86	0.00	789.86	750.00	1.053
21	106.0	0.21	788.37	0.00	788.37	750.00	1.051
22	105.8	0.21	786.89	0.00	786.89	750.00	1.049
23	105.6	0.21	785.40	0.00	785.40	750.00	1.047
24	105.4	0.21	783.91	0.00	783.91	750.00	1.045
25	105.2	0.21	782.42	0.00	782.42	750.00	1.043

26	105.0	0.21	780.94	0.00	780.94	750.00	1.041
27	104.8	0.21	779.45	0.00	779.45	750.00	1.039
28	104.6	0.21	777.96	0.00	777.96	750.00	1.037
29	104.4	0.21	776.47	0.00	776.47	750.00	1.035
30	104.2	0.21	774.99	0.00	774.99	750.00	1.033
31	104.0	0.21	773.50	0.00	773.50	750.00	1.031
32	103.8	0.21	772.01	0.00	772.01	750.00	1.029
33	103.6	0.21	770.52	0.00	770.52	750.00	1.027
34	103.4	0.21	769.04	0.00	769.04	750.00	1.025
35	103.2	0.21	767.55	0.00	767.55	750.00	1.023
36	103.0	0.21	766.06	0.00	766.06	750.00	1.021
37	102.8	0.21	764.57	0.00	764.57	750.00	1.019
38	102.6	0.21	763.09	0.00	763.09	750.00	1.017
39	102.4	0.20	761.60	0.00	761.60	750.00	1.015
40	102.2	0.20	760.11	0.00	760.11	750.00	1.013
41	102.0	0.20	758.62	0.00	758.62	750.00	1.011
42	101.8	0.20	757.14	0.00	757.14	750.00	1.010
43	101.6	0.20	755.65	0.00	755.65	750.00	1.008
44	101.4	0.20	754.16	0.00	754.16	750.00	1.006
45	101.2	0.20	752.67	0.00	752.67	750.00	1.004
46	101.0	0.20	751.19	0.00	751.19	750.00	1.002
47	100.8	0.20	749.70	0.00	749.70	750.00	1.000

Final value of x is 100.80 mm and flexural tension reinforcement area is 1500.00 mm2
Working depth of reinforcement $d = 500.00 \text{ mm}$

Strength reduction factor for reinforcement Class N in bending without axial tension or compression (Table 2.2.2)

$$\phi = 1.19 - 13 \cdot k_{uo}/12 = 1.19 - 13 \cdot 0.20/12 = 0.97$$

$$\phi > 0.8 \rightarrow \phi = 0.8$$

Check maximum allowable depth of the rectangular compression block (8.1.5)

$$a = \gamma \cdot x = 0.70 \cdot 100.80 = 70.56 \text{ mm} \leq a_{max} = \gamma \cdot k_u \cdot d_0 = 0.70 \cdot 0.36 \cdot 500 = 126.00 \text{ mm}$$

2. Calculation moment resistance M_d (2.2.2)

$$\phi M_d = (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) \cdot \phi = (49.12 + 0.00 + 299.40) \cdot 0.80 = 278.82 \text{ kN-m}$$

$$M^* = 278.82 \text{ kN-m} > M_d = 278.82 \text{ kN-m (Ratio: 1.000)}$$

STATUS NG!
Ratio: 1.000

3. Minimum required flexural tension reinforcement in a beam section (8.1.6.1(2))

$$\alpha_b = 0.2$$

$$f'_{ct,f} = 0.6\sqrt{f'_c} = 0.6\sqrt{50} = 4.24$$

$$A_{st,min} = \alpha_b \cdot (\frac{h}{d} \cdot \frac{f'_{ct,f}}{f_{sy}}) \cdot b_w \cdot d = 0.20 \cdot (\frac{550}{500.00}) \cdot \frac{4.24}{500} \cdot 250 \cdot 500.00 = 233.35 \text{ mm}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$A_{st,max} = 0.04 \cdot b \cdot d = 0.04 \cdot 250 \cdot 500.00 = 5000.00 \text{ mm}^2$$

5. Check of required flexural tension reinforcement in a beam section

$$A_{st} = 1500.00 \text{ mm}^2 \leq A_{st,max} = 5000.00 \text{ mm}^2 \text{ (Ratio: 0.300)}$$

STATUS OK!
Ratio: 0.300

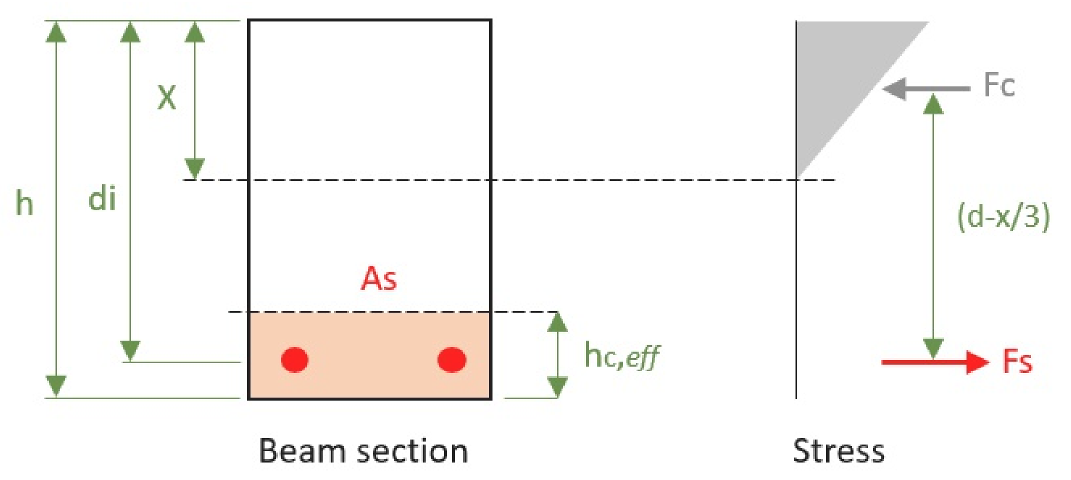
$$A_{st} = 1500.00 \text{ mm}^2 \geq A_{st,min} = 233.35 \text{ mm}^2 \text{ (Ratio: 0.156)}$$

STATUS OK!
Ratio: 0.156

Crack control check (Positive bending moment case)

3.1.8.3, 8.6.1

CRACK CONTROL OF BEAMS



Section input data:
Section concrete area $A_g = 137500 \text{ mm}^2$
Web width $b_w = 250 \text{ mm}$
Age of concrete at loading $\tau = 3 \text{ days}$
Age of concrete at the moment considered $t = 10000 \text{ days}$
Environment type factor $k_4 = 0.7$
Modulus of elasticity of concrete $E_c = 34800.00 \text{ MPa}$
Given bending moment $M_s^* = 150.00 \text{ kN-m}$

1. Design creep coefficient ϕ_{cc} (3.1.8.3)

$$\text{Basic creep coefficient } \phi_{cc,b} = 2.40$$

Notional size of the member

$$t_h = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 137500.00}{1600} = 171.88 \text{ mm}$$

$$\alpha_2 = 1.0 + 1.12 \cdot e^{-0.008 \cdot t_h} = 1.0 + 1.12 \cdot e^{-0.008 \cdot 171.88} = 1.28$$

$$k_2 = \frac{\alpha_2 \cdot t^{0.8}}{t^{0.8} + 0.15 \cdot t_h} = \frac{1.28 \cdot 10000^{0.8}}{10000^{0.8} + 0.15 \cdot 171.88} = 1.26$$

$$k_3 = 2.7/\left[1.0 + log(\tau)\right] = 2.7/\left[1.0 + log(3)\right] = 1.29$$

$$\acute{f_c} \leq 50 \rightarrow k_5 = 1.0$$

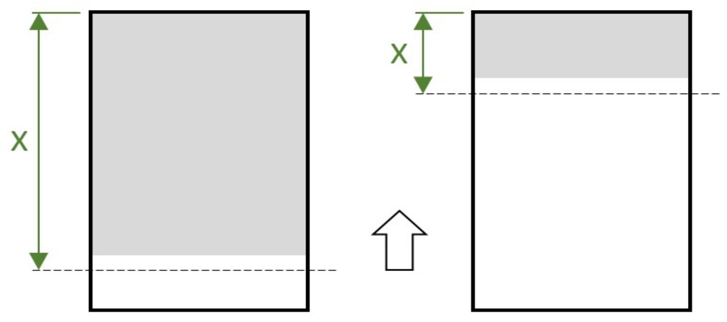
$$\phi_{cc} = k_2 \cdot k_3 \cdot k_4 \cdot k_5 \cdot \phi_{cc,b} = 1.26 \cdot 1.29 \cdot 0.70 \cdot 1.00 \cdot 2.40 = 2.73$$

Effective modulus

$$E_{eff} = \frac{E_{cm}}{1 + \phi_{cc}} = \frac{34800.00}{1 + 2.73} = 9331.99 \text{ MPa}$$

$$a_e = \frac{E_s}{E_{eff}} = \frac{200000}{9331.99} = 21.43$$

2. Calculation of neutral axis depth x



Calculation is based on iterative process:
- Assume x
- Calculate left part of force equilibrium $A_{comp.} \cdot \frac{x}{2} + \sum a_e \cdot A_s \cdot \acute{d_i} + \sum a_e \cdot A_s \cdot d_i$
- Calculate right part of force equilibrium $A_{comp.} + a_e \cdot A_s + a_e \cdot \acute{A_s}$

Searching of neutral axis x (from 500 to 0 mm)

Iter.	x (mm)	As (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	500.00	0.00	47323.74	78573.74	1.660
2	490.00	1500.00	46086.24	75777.27	1.644
3	480.00	1500.00	44873.74	73030.79	1.627
4	470.00	1500.00	43686.24	70334.32	1.610
5	460.00	1500.00	42523.74	67687.84	1.592
6	450.00	1500.00	41386.24	65091.37	1.573
7	440.00	1500.00	40273.74	62544.89	1.553
8	430.00	1500.00	39186.24	60048.42	1.532
9	420.00	1500.00	38123.74	57601.94	1.511
10	410.00	1500.00	37086.24	55205.47	1.489
11	400.00	1500.00	36073.74	52858.99	1.465
12	390.00	1500.00	35086.24	50562.52	1.441
13	380.00	1500.00	34123.74	48316.04	1.416
14	370.00	1500.00	33186.24	46119.57	1.390
15	360.00	1500.00	32273.74	43973.09	1.363
16	350.00	1500.00	31386.24	41876.62	1.334
17	340.00	1500.00	30523.74	39830.14	1.305
18	330.00	1500.00	29686.24	37833.67	1.274
19	320.00	1500.00	28873.74	35887.20	1.243
20	310.00	1500.00	28086.24	33990.72	1.210
21	300.00	1500.00	27323.74	32144.25	1.176
22	290.00	1500.00	26586.24	30347.77	1.141
23	280.00	1500.00	25873.74	28601.30	1.105
24	270.00	1500.00	25186.24	26904.82	1.068
25	260.00	1500.00	24523.74	25258.35	1.030
left part < right part. Updating of iterations					
1	250.00	1500.00	23886.24	23661.87	0.991
2	259.80	1500.00	24510.75	25225.93	1.029
3	259.60	1500.00	24497.76	25193.53	1.028
4	259.40	1500.00	24484.79	25161.15	1.028
5	259.20	1500.00	24471.82	25128.79	1.027
6	259.00	1500.00	24458.87	25096.45	1.026
7	258.80	1500.00	24445.92	25064.13	1.025
8	258.60	1500.00	24432.99	25031.83	1.025

9	258.40	1500.00	24420.06	24999.55	1.024
10	258.20	1500.00	24407.15	24967.29	1.023
11	258.00	1500.00	24394.24	24935.05	1.022
12	257.80	1500.00	24381.35	24902.83	1.021
13	257.60	1500.00	24368.46	24870.63	1.021
14	257.40	1500.00	24355.59	24838.45	1.020
15	257.20	1500.00	24342.72	24806.29	1.019
16	257.00	1500.00	24329.87	24774.15	1.018
17	256.80	1500.00	24317.02	24742.03	1.017
18	256.60	1500.00	24304.19	24709.93	1.017
19	256.40	1500.00	24291.36	24677.86	1.016
20	256.20	1500.00	24278.55	24645.80	1.015
21	256.00	1500.00	24265.74	24613.76	1.014
22	255.80	1500.00	24252.95	24581.74	1.014
23	255.60	1500.00	24240.16	24549.74	1.013
24	255.40	1500.00	24227.39	24517.76	1.012
25	255.20	1500.00	24214.62	24485.80	1.011
26	255.00	1500.00	24201.87	24453.86	1.010
27	254.80	1500.00	24189.12	24421.94	1.010
28	254.60	1500.00	24176.39	24390.04	1.009
29	254.40	1500.00	24163.66	24358.16	1.008
30	254.20	1500.00	24150.95	24326.30	1.007
31	254.00	1500.00	24138.24	24294.46	1.006
32	253.80	1500.00	24125.55	24262.64	1.006
33	253.60	1500.00	24112.86	24230.84	1.005
34	253.40	1500.00	24100.19	24199.06	1.004
35	253.20	1500.00	24087.52	24167.30	1.003
36	253.00	1500.00	24074.87	24135.56	1.003
37	252.80	1500.00	24062.22	24103.84	1.002
38	252.60	1500.00	24049.59	24072.14	1.001
39	252.40	1500.00	24036.96	24040.47	1.000
40	252.20	1500.00	24024.35	24008.81	0.999

Final value of x is 252.20 mm and tensioning rebar area is $1500.00 \mathrm{~mm}^2$
Working depth of reinforcement $d = 500.00 \mathrm{~mm}$

3. Calculation of stress in tensionng zone of reinforcement

$$\sigma_{scr} = \frac{M^*}{A_s \cdot (d - x/3)} = \frac{150000000}{1500.00 \cdot (500.00 - 252.20/3)} = 240.42 \mathrm{~MPa}$$

4. Limiting stress check (8.6.1)

For nominal bar diameter in tension zone d_b
$= 30.902$ mm the maximum stress is 166.86 MPa (Table 8.6.1(A))

For centre-to-centre spacing in tension zone 75.00 mm the maximum stress is 340.00 MPa (Table 8.6.1(B))

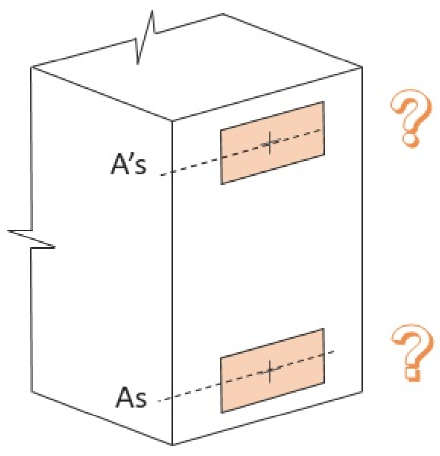
STATUS NG!
Ratio: 1.441

$\sigma_{scr} = 240.42 \mathrm{~MPa} > 166.86 \mathrm{~MPa}$ (For nominal bar diameter) (Ratio: 1.441)

$\sigma_{scr} = 240.42 \mathrm{~MPa} \leq 340.00 \mathrm{~MPa}$ (For centre-to-centre spacing) (Ratio: 0.707)

STATUS OK!
Ratio: 0.707

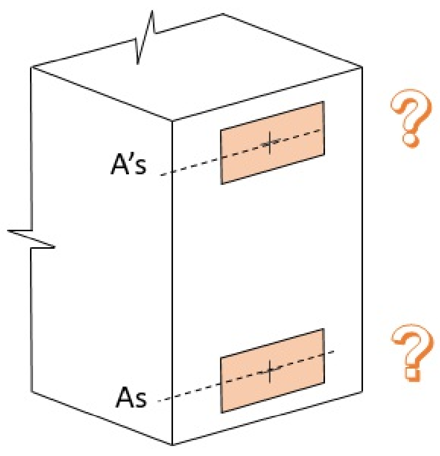
Flexure check (Negative bending moment case)



Bottom Reinforcement is absent in the section. Design checks can't be performed. But as acting moment value is equal to zero no need to check.

STATUS OK!

Crack control check (Negative bending moment case)



Bottom Reinforcement is absent in the section. Design checks can't be performed. But as acting moment value is equal to zero no need to check.

STATUS OK!

Shear check

8.2.7.1, 8.2.6, 8.2.9

SHEAR FORCE CAPACITY (Members without shear reinforcement)

Section input data:
Mean width of web $b_w = 250\ \mathrm{mm}$
Section concrete area $A_g = 137500\ \mathrm{mm}^2$
Tensioning rebar area $A_{st} = 1500.00\ \mathrm{mm}^2$
Working depth of reinforcement $d_0 = 500\ \mathrm{mm}$
Given shear force $V^* = 0.00\ \mathrm{kN}$
Corresponding axial force $N^* = 0.00\ \mathrm{kN}$

1. Determine concrete shear capacity (8.2.7.1)

$$\acute{f_{cv}} = (\acute{f_c})^{1/3} = (50)^{1/3} = 3.68\ \mathrm{MPa} \leq 4\ \mathrm{MPa}$$

$$\beta_1 = 1.1 \cdot (1.6 - \frac{d_0}{1000}) = 1.1 \cdot (1.6 - \frac{500}{1000}) = 1.21 \geq 1.1$$

$$\beta_2 = 1.0$$

$$\beta_3 = 1.0$$

$$V_{uc} = \beta_1 \cdot \beta_2 \cdot \beta_3 \cdot b_w \cdot d_0 \cdot \acute{f_{cv}} \cdot (\frac{A_{st}}{b_w \cdot d_0})^{1/3} = 1.21 \cdot 1.00 \cdot 1.00 \cdot 250 \cdot 500 \cdot 3.68 \cdot (\frac{1500.00}{250 \cdot 500})^{1/3} = 127.57\ \mathrm{kN}$$

$$V^* = 0\ \mathrm{kN} \leq V_{cv} = 127.57\ \mathrm{kN}$$

STATUS OK!