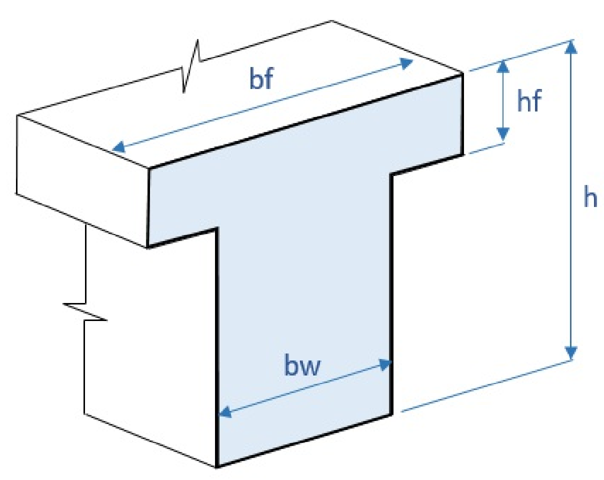
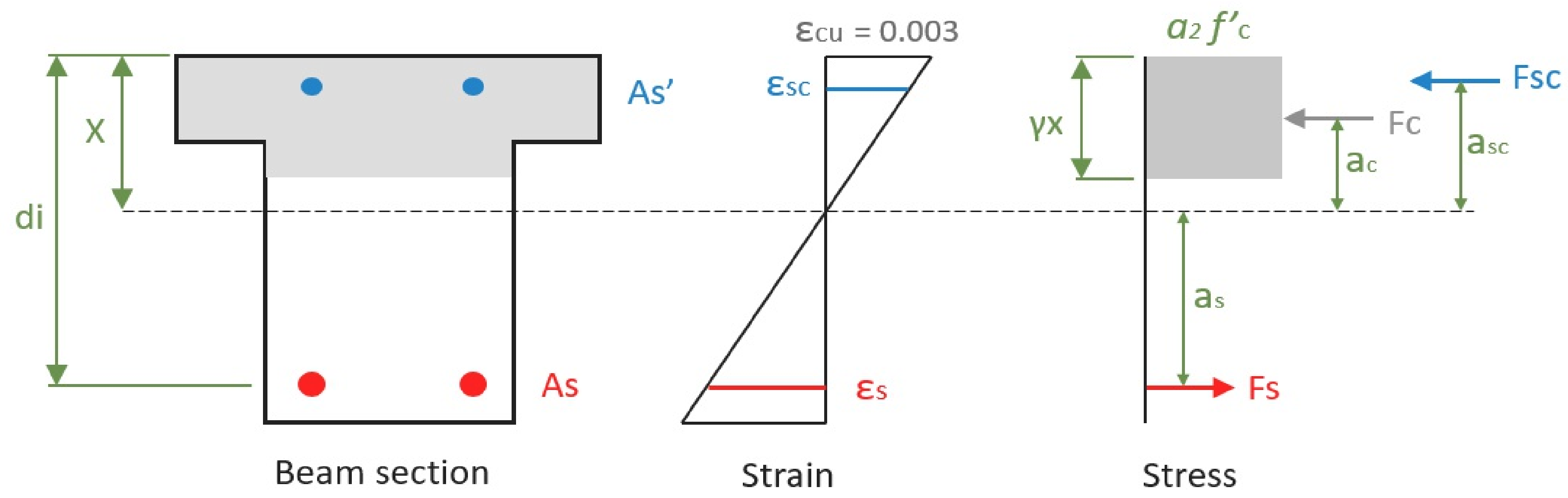
REFERENCES	CALCULATIONS	RESULTS
Code: AS 3600-2009	**MEMBER #1 (SECTION POSITION 2500 mm) BEAM DESIGN REPORT** **Project details** Project Name: Project ID: Company: Designer: Client: Project Notes: Project Units: metric **General member design information** Dimentions:  Height $h = 450\ \mathrm{mm}$ Flange width $b_f = 800\ \mathrm{mm}$ Flange thickness $h_f = 120\ \mathrm{mm}$ Web width $b_w = 450\ \mathrm{mm}$ Member length = $5000\ \mathrm{mm}$ Material properties: Concrete strength $\acute{f_c} = 25\ \mathrm{MPa}$ 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 = $250\ \mathrm{kN\text{-}m}$ For shear force in section: LC1: USER = $4500\ \mathrm{kN}$ **Load Combinations (Serviceability Limit State)** For bending moment in section: LC1: USER = $25\ \mathrm{kN\text{-}m}$	
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 $d_0 = 380\ \mathrm{mm}$ Given bending moment $M^* = 250\ \mathrm{kN\text{-}m}$ $\mathrm{Section\ Rebar}$	

Depth di (mm)	bar diameter (mm)	bar area Asi (mm2)
380	25	490.87
380	25	490.87
380	25	490.87
380	25	490.87
50	16	201.06
50	16	201.06

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

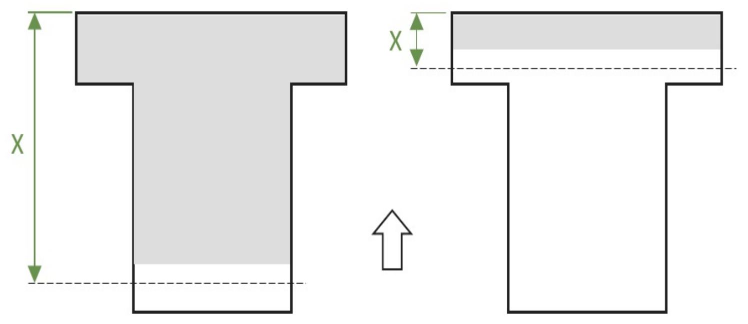
$$\alpha_2 = 1.0 - 0.003 \cdot f_c' = 1.0 - 0.003 \cdot 25 = 0.93$$

$$\alpha_2 > 0.85 \rightarrow \alpha_2 = 0.85$$

$$\gamma = 1.05 - 0.007 \cdot f_c' = 1.05 - 0.007 \cdot 25 = 0.88$$

$$\gamma > 0.85 \rightarrow \gamma = 0.85$$

1. Calculation of neutral axis depth x



Calculation is based on iterative process:
- Assume x
- Calculate concrete force $F_c = \alpha_2 \cdot f_c' \cdot \int_{dA} \cdot \gamma \cdot x$
- Calculate compression force in steel $F_{cs} = \sum A_{s,i}' \cdot f_{s,i}$
- 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 380 to 0 mm)

Iter.	x (mm)	kuo = x/do	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	380.0	1.00	3981.19	201.06	4182.25	0.00	Infinity
2	370.5	0.97	3903.97	201.06	4105.03	30.21	135.89
3	361.0	0.95	3826.75	201.06	4027.81	62.00	64.96
4	351.5	0.93	3749.54	201.06	3950.60	95.52	41.36
5	342.0	0.90	3672.32	201.06	3873.38	130.90	29.59
6	332.5	0.88	3595.10	201.06	3796.16	168.30	22.56
7	323.0	0.85	3517.88	201.06	3718.94	207.90	17.89
8	313.5	0.82	3440.67	201.06	3641.73	249.90	14.57
9	304.0	0.80	3363.45	201.06	3564.51	294.52	12.10
10	294.5	0.78	3286.23	200.31	3486.54	342.03	10.19
11	285.0	0.75	3209.02	198.94	3407.96	392.70	8.68
12	275.5	0.72	3131.80	197.48	3329.28	446.86	7.45
13	266.0	0.70	3054.58	195.92	3250.50	504.89	6.44
14	256.5	0.68	2977.36	194.24	3171.60	567.23	5.59
15	247.0	0.65	2900.15	192.43	3092.58	634.36	4.88

16	237.5	0.63	2822.93	190.48	3013.41	706.85	4.26
17	228.0	0.60	2745.71	188.36	2934.07	785.39	3.74
18	218.5	0.57	2668.50	186.06	2854.56	870.76	3.28
19	209.0	0.55	2591.28	183.55	2774.83	963.89	2.88
20	199.5	0.53	2514.06	180.80	2694.86	981.74	2.74
21	190.0	0.50	2436.84	177.78	2614.62	981.74	2.66
22	180.5	0.47	2359.63	174.44	2534.06	981.74	2.58
23	171.0	0.45	2282.41	170.72	2453.13	981.74	2.50
24	161.5	0.42	2205.19	166.57	2371.77	981.74	2.42
25	152.0	0.40	2127.97	161.91	2289.88	981.74	2.33
26	142.5	0.38	2050.76	156.62	2207.37	981.74	2.25
27	133.0	0.35	1921.85	150.57	2072.42	981.74	2.11
28	123.5	0.33	1784.58	143.59	1928.17	981.74	1.96
29	114.0	0.30	1647.30	135.45	1782.75	981.74	1.82
30	104.5	0.28	1510.03	125.83	1635.86	981.74	1.67
31	95.0	0.25	1372.75	114.29	1487.04	981.74	1.51
32	85.5	0.23	1235.47	100.18	1335.65	981.74	1.36
33	76.0	0.20	1098.20	82.54	1180.74	981.74	1.20
34	66.5	0.17	960.92	59.86	1020.79	981.74	1.04
(Fc + Fcs) < Fs. Updating of iterations							
1	57.0	0.15	823.65	29.63	853.28	981.74	0.87
2	66.3	0.17	957.49	59.21	1016.71	981.74	1.04
3	66.0	0.17	954.06	58.56	1012.62	981.74	1.03
4	65.8	0.17	950.63	57.90	1008.53	981.74	1.03
5	65.6	0.17	947.20	57.24	1004.43	981.74	1.02
6	65.3	0.17	943.77	56.57	1000.33	981.74	1.02
7	65.1	0.17	940.33	55.89	996.23	981.74	1.01
8	64.8	0.17	936.90	55.21	992.11	981.74	1.01
9	64.6	0.17	933.47	54.53	988.00	981.74	1.01
10	64.4	0.17	930.04	53.84	983.88	981.74	1.00
11	64.1	0.17	926.61	53.15	979.75	981.74	1.00

Final value of x is 64.13 mm and flexural tension reinforcement area is 1963.48 mm2

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

$$\phi = 1.19 - 13 \cdot k_{uo}/12 = 1.19 - 13 \cdot 0.17/12 = 1.01$$

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

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

$$a = \gamma \cdot x = 0.85 \cdot 64.13 = 54.51 \text{ mm} \leq a_{max} = \gamma \cdot k_u \cdot d_0 = 0.85 \cdot 0.36 \cdot 380 = 116.28 \text{ mm}$$

2. Calculation moment resistance M_d (2.2.2)

$$\phi M_d = (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) \cdot \phi = (34.17 + 0.75 + 310.11) \cdot 0.80 = 276.02 \text{ kN-m}$$

$$M^* = 250.00 \text{ kN-m} \leq M_d = 276.02 \text{ kN-m}$$

STATUS OK!

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

$$f'_{ct,f} = 0.6\sqrt{f'_c} = 0.6\sqrt{25} = 3.00$$

for T-Sections with the flange in tension

$$\alpha_b = 0.2 + (\frac{b_f}{b_w} - 1) \cdot (0.25 \cdot (\frac{h_f}{h}) - 0.08) = 0.2 + (\frac{800}{450} - 1) \cdot (0.25 \cdot (\frac{120}{450}) - 0.08) = 0.19$$

$$\alpha_b < 0.2 \cdot (\frac{b_f}{b_w})^{2/3} = 0.2 \cdot (\frac{800}{450})^{2/3} = 0.29$$

$$\alpha_b = 0.29$$

$$A_{st,min} = \alpha_b \cdot (\frac{h}{d_0}) \cdot \frac{f'_{ct,f}}{f_{sy}} \cdot b_w \cdot d_0 = 0.29 \cdot (\frac{450}{380}) \cdot \frac{3.00}{500} \cdot 450 \cdot 380 = 356.47 \text{ mm}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$A_{st,max} = 0.04 \cdot b_w \cdot d = 0.04 \cdot 450 \cdot 380 = 6840.00 \text{ mm}^2$$

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

$$A_{st} = 1963.48 \text{ mm}^2 \leq A_{st,max} = 6840.00 \text{ mm}^2$$

STATUS OK!

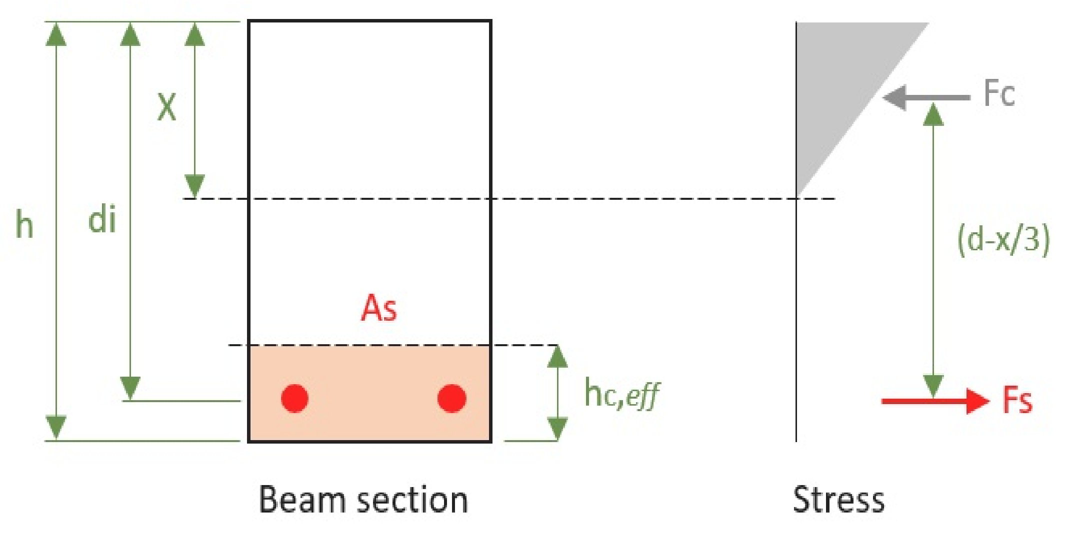
$$A_{st} = 1963.48 \text{ mm}^2 \geq A_{st,min} = 356.47 \text{ mm}^2$$

STATUS OK!

Crack control check (Positive bending moment case)

3.1.8.3, 8.6.1

CRACK CONTROL OF BEAMS



Section input data:
Section concrete area $A_g = 244500 \text{ mm}^2$
Web width $b_w = 450 \text{ mm}$
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 $E_c = 26700.00 \text{ MPa}$
Given bending moment $M_s^* = 25 \text{ kN-m}$

Section Rebar

Depth di (mm)	bar diameter (mm)	bar area Asi (mm2)
380	25	490.87
380	25	490.87
380	25	490.87
380	25	490.87
50	16	201.06
50	16	201.06

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

$$\text{Basic creep coefficient } \phi_{cc,b} = 4.20$$

Notional size of the member

$$t_h = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 244500.00}{2500} = 195.60 \text{ mm}$$

$$\alpha_2 = 1.0 + 1.12 \cdot e^{-0.008 \cdot t_h} = 1.0 + 1.12 \cdot e^{-0.008 \cdot 195.60} = 1.23$$

$$k_2 = \frac{\alpha_2 \cdot t^{0.8}}{t^{0.8} + 0.15 \cdot t_h} = \frac{1.23 \cdot 10000^{0.8}}{10000^{0.8} + 0.15 \cdot 195.60} = 1.21$$

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

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

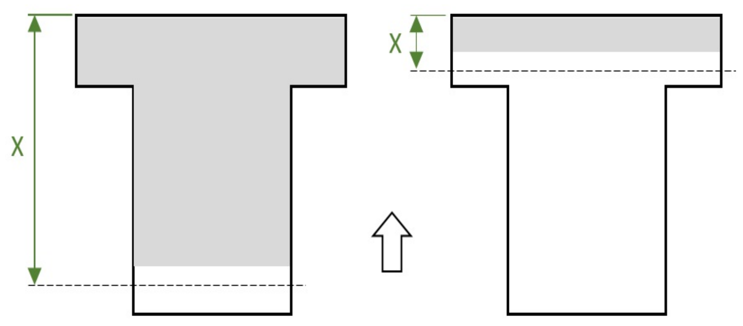
$$\phi_{cc} = k_2 \cdot k_3 \cdot k_4 \cdot k_5 \cdot \phi_{cc,b} = 1.21 \cdot 1.29 \cdot 0.70 \cdot 1.00 \cdot 4.20 = 4.58$$

Effective modulus

$$E_{eff} = \frac{E_{cm}}{1 + \phi_{cc}} = \frac{26700.00}{1 + 4.58} = 4781.86 \text{ MPa}$$

$$a_e = \frac{E_s}{E_{eff}} = \frac{200000}{4781.86} = 41.82$$

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 left part of force equilibrium $A_{comp.} + a_e \cdot A_s + a_e \cdot \acute{A_s}$

Searching of neutral axis x (from 380 to 0 mm)

Iter.	x (mm)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	380.00	67057.31	118537.44	1.77
2	370.50	65453.12	113990.11	1.74
3	361.00	63889.54	109524.02	1.71
4	351.50	62366.57	105139.14	1.69
5	342.00	60884.21	100835.49	1.66
6	332.50	59442.47	96613.07	1.63
7	323.00	58041.34	92471.87	1.59
8	313.50	56680.82	88411.90	1.56
9	304.00	55360.91	84433.15	1.53
10	294.50	54081.62	80535.63	1.49
11	285.00	52842.94	76719.33	1.45
12	275.50	51644.87	72984.26	1.41
13	266.00	50487.41	69330.41	1.37
14	256.50	49370.57	65757.78	1.33
15	247.00	48294.34	62266.38	1.29
16	237.50	47258.72	58856.21	1.25
17	228.00	46263.71	55527.26	1.20
18	218.50	45309.32	52279.54	1.15
19	209.00	44395.54	49113.04	1.11
20	199.50	43522.37	46027.77	1.06
21	190.00	42689.81	43023.72	1.01
left part < right part. Updating of iterations				
1	180.50	41897.87	40100.90	0.96
2	189.76	42669.52	42949.66	1.01
3	189.52	42649.25	42875.65	1.01
4	189.29	42629.01	42801.69	1.00
5	189.05	42608.79	42727.78	1.00
6	188.81	42588.60	42653.92	1.00
7	188.57	42568.43	42580.12	1.00
8	188.34	42548.29	42506.36	1.00

Final value of x is 188.34 mm and tensioning rebar Area is 1963.48 mm2

3. Calculation of stress in tensionng zone of reinforcement

$$\sigma_{scr} = \frac{M^*}{A_s \cdot (d - x/3)} = \frac{25000000}{1963.48 \cdot (380 - 188.34/3)} = 40.14 \text{ MPa}$$

4. Limiting stress check (8.6.1)

$$\text{For nominal bar diameter in tension zone } d_b = 25 \text{ mm the maximum stress is } 203.75 \text{ MPa (Table 8.6.1(A))}$$

$$\text{For centre-to-centre spacing in tension zone } 87.50 \text{ mm the maximum stress is } 330.00 \text{ MPa (Table 8.6.1(B))}$$

$$\sigma_{scr} = 40.14 \text{ MPa} \leq 203.75 \text{ MPa (For nominal bar diameter)}$$

STATUS OK!

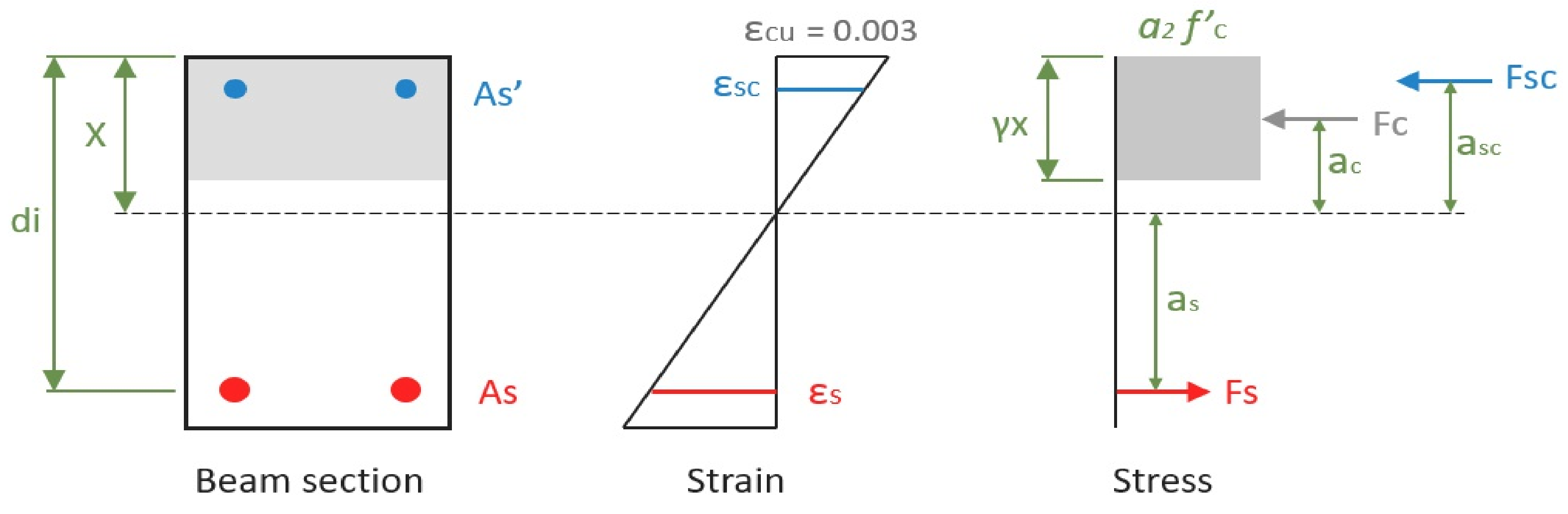
$\sigma_{scr} = 40.14 \text{ MPa} \leq 330.00 \text{ MPa (For centre-to-centre spacing)}$

STATUS OK!

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 (Negative 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 $d_0 = 400 \text{ mm}$
Given bending moment $M^* = 0 \text{ kN-m}$

Section Rebar

Depth di (mm)	bar diameter (mm)	bar area Asi (mm2)
400	16	201.06
400	16	201.06
70	25	490.87
70	25	490.87
70	25	490.87
70	25	490.87

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

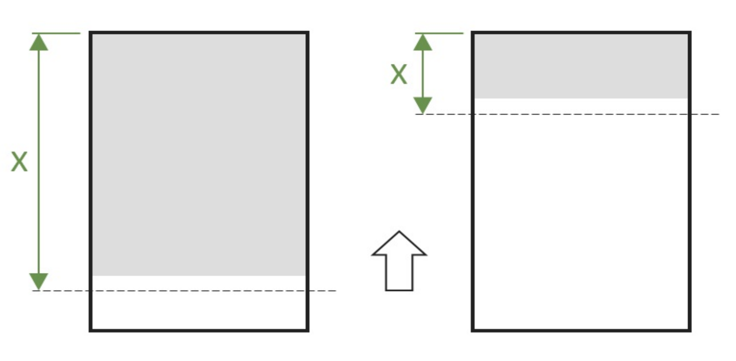
$$\alpha_2 = 1.0 - 0.003 \cdot \acute{f_c} = 1.0 - 0.003 \cdot 25 = 0.93$$

$$\alpha_2 > 0.85 \rightarrow \alpha_2 = 0.85$$

$$\gamma = 1.05 - 0.007 \cdot \acute{f_c} = 1.05 - 0.007 \cdot 25 = 0.88$$

$$\gamma > 0.85 \rightarrow \gamma = 0.85$$

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 400 to 0 mm)

Iter.	x (mm)	kuo = x/do	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio

1	400.0	1.00	4143.75	971.92	5115.67	0.00	Infinity
2	390.0	0.97	4062.47	966.64	5029.11	6.19	812.92
3	380.0	0.95	3981.19	961.07	4942.26	12.70	389.20
4	370.0	0.93	3899.91	955.21	4855.11	19.56	248.18
5	360.0	0.90	3818.63	949.02	4767.64	26.81	177.84
6	350.0	0.88	3737.34	942.47	4679.81	34.47	135.77
7	340.0	0.85	3656.06	935.54	4591.60	42.58	107.84
8	330.0	0.82	3574.78	928.19	4502.97	51.18	87.98
9	320.0	0.80	3493.50	920.38	4413.88	60.32	73.18
10	310.0	0.78	3412.22	912.07	4324.29	70.05	61.73
11	300.0	0.75	3330.94	903.20	4234.14	80.42	52.65
12	290.0	0.72	3249.66	893.72	4143.38	91.52	45.27
13	280.0	0.70	3168.38	883.57	4051.94	103.40	39.19
14	270.0	0.68	3087.09	872.66	3959.75	116.17	34.09
15	260.0	0.65	3005.81	860.91	3866.72	129.92	29.76
16	250.0	0.63	2924.53	848.22	3772.75	144.76	26.06
17	240.0	0.60	2843.25	834.48	3677.73	160.85	22.86
18	230.0	0.57	2761.97	819.54	3581.51	178.33	20.08
19	220.0	0.55	2680.69	803.24	3483.93	197.40	17.65
20	210.0	0.53	2599.41	785.39	3384.80	201.06	16.83
21	200.0	0.50	2518.13	765.76	3283.88	201.06	16.33
22	190.0	0.47	2436.84	744.06	3180.90	201.06	15.82
23	180.0	0.45	2355.56	719.94	3075.51	201.06	15.30
24	170.0	0.42	2274.28	692.99	2967.27	201.06	14.76
25	160.0	0.40	2193.00	662.67	2855.67	201.06	14.20
26	150.0	0.38	2111.72	628.31	2740.03	201.06	13.63
27	140.0	0.35	2023.00	589.04	2612.04	201.06	12.99
28	130.0	0.33	1878.50	543.73	2422.23	201.06	12.05
29	120.0	0.30	1734.00	490.87	2224.87	201.06	11.07
30	110.0	0.28	1589.50	428.40	2017.90	201.06	10.04
31	100.0	0.25	1445.00	353.43	1798.43	201.06	8.94
32	90.0	0.23	1300.50	261.80	1562.30	201.06	7.77
33	80.0	0.20	1156.00	147.26	1303.26	201.06	6.48
34	70.0	0.17	1011.50	0.00	1011.50	201.06	5.03
35	60.0	0.15	867.00	0.00	867.00	397.41	2.18
36	50.0	0.13	722.50	0.00	722.50	672.30	1.07
(Fc + Fcs) < Fs. Updating of iterations							
1	40.0	0.10	578.00	0.00	578.00	1084.63	0.53
2	49.8	0.12	718.89	0.00	718.89	680.58	1.06
3	49.5	0.12	715.27	0.00	715.27	688.96	1.04
4	49.3	0.12	711.66	0.00	711.66	697.41	1.02
5	49.0	0.12	708.05	0.00	708.05	705.95	1.00
6	48.8	0.12	704.44	0.00	704.44	714.59	0.99

Final value of x is 48.75 mm and flexural tension reinforcement area is 2365.60 mm2

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

$$\phi = 1.19 - 13 \cdot k_{uo}/12 = 1.19 - 13 \cdot 0.12/12 = 1.06$$

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

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

$$a = \gamma \cdot x = 0.85 \cdot 48.75 = 41.44 \text{ mm} \leq a_{max} = \gamma \cdot k_u \cdot d_0 = 0.85 \cdot 0.36 \cdot 400 = 122.40 \text{ mm}$$

2. Calculation moment resistance M_d (2.2.2)

$$\phi M_d = (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) \cdot \phi = (19.75 + 0.00 + 81.53) \cdot 0.80 = 81.02 \text{ kN-m}$$

$$M^* = 0.00 \text{ kN-m} \leq M_d = 81.02 \text{ kN-m}$$

STATUS OK!

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

$$f'_{ct,f} = 0.6\sqrt{f'_c} = 0.6\sqrt{25} = 3.00$$

$$\text{for T-Sections with the flange in tension}$$

$$\alpha_b = 0.2 + (\frac{b_f}{b_w} - 1) \cdot (0.25 \cdot (\frac{h_f}{h}) - 0.08) = 0.2 + (\frac{800}{450} - 1) \cdot (0.25 \cdot (\frac{120}{450}) - 0.08) = 0.19$$

$$\alpha_b < 0.2 \cdot (\frac{b_f}{b_w})^{2/3} = 0.2 \cdot (\frac{800}{450})^{2/3} = 0.29$$

$$\alpha_b = 0.29$$

$$A_{st,min} = \alpha_b \cdot (\frac{h}{d_0}) \cdot \frac{f'_{ct,f}}{f_{sy}} \cdot b_w \cdot d_0 = 0.29 \cdot (\frac{450}{400}) \cdot \frac{3.00}{500} \cdot 450 \cdot 400 = 356.47 \text{ mm}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$A_{st,max} = 0.04 \cdot b_w \cdot d = 0.04 \cdot 450 \cdot 400 = 7200.00 \text{ mm}^2$$

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

$$A_{st} = 2365.60 \text{ mm}^2 \leq A_{st,max} = 7200.00 \text{ mm}^2$$

STATUS OK!

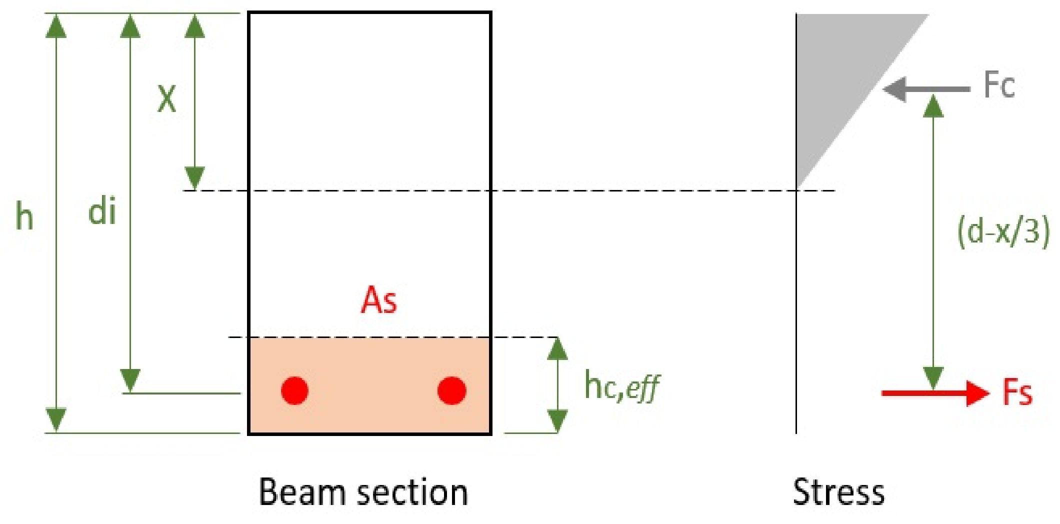
$$A_{st} = 2365.60 \text{ mm}^2 \geq A_{st,min} = 356.47 \text{ mm}^2$$

STATUS OK!

Crack control check (Negative bending moment case)

3.1.8.3, 8.6.1

CRACK CONTROL OF BEAMS



Section input data:

Section concrete area $A_g = 244500\ \mathrm{mm}^2$
Web width $b_w = 450\ \mathrm{mm}$
Age of concrete at loading $\tau = 3$ days
Age of concrete at the moment considered $t = 10000$ days
Environment type factor $k_4 = 0.7$
Modulus of elasticity of concrete $E_c = 26700.00\ \mathrm{MPa}$
Given bending moment $M_s^* = 0\ \mathrm{kN\text{-}m}$

Section Rebar

Depth di (mm)	bar diameter (mm)	bar area Asi (mm2)
400	16	201.06
400	16	201.06
70	25	490.87
70	25	490.87
70	25	490.87
70	25	490.87

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

$$\text{Basic creep coefficient } \phi_{cc,b} = 4.20$$

Notional size of the member

$$t_h = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 244500.00}{2500} = 195.60\ \mathrm{mm}$$

$$\alpha_2 = 1.0 + 1.12 \cdot e^{-0.008 \cdot t_h} = 1.0 + 1.12 \cdot e^{-0.008 \cdot 195.60} = 1.23$$

$$k_2 = \frac{\alpha_2 \cdot t^{0.8}}{t^{0.8} + 0.15 \cdot t_h} = \frac{1.23 \cdot 10000^{0.8}}{10000^{0.8} + 0.15 \cdot 195.60} = 1.21$$

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

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

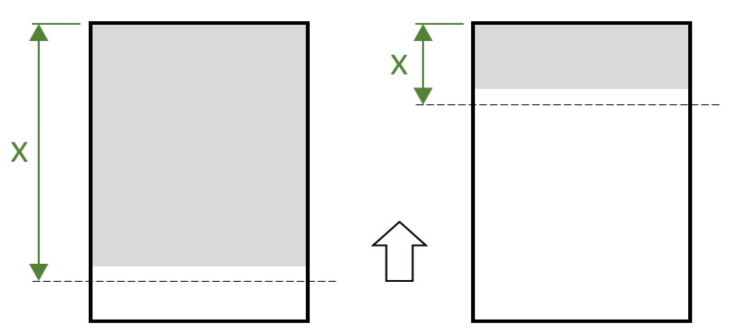
$$\phi_{cc} = k_2 \cdot k_3 \cdot k_4 \cdot k_5 \cdot \phi_{cc,b} = 1.21 \cdot 1.29 \cdot 0.70 \cdot 1.00 \cdot 4.20 = 4.58$$

Effective modulus

$$E_{eff} = \frac{E_{cm}}{1 + \phi_{cc}} = \frac{26700.00}{1 + 4.58} = 4781.86\ \mathrm{MPa}$$

$$a_e = \frac{E_s}{E_{eff}} = \frac{200000}{4781.86} = 41.82$$

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 left part of force equilibrium $A_{comp.} + a_e \cdot A_s + a_e \cdot \acute{A_s}$

Searching of neutral axis x (from 400 to 0 mm)

Iter.	x (mm)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	400.00	50995.97	128376.25	2.52
2	390.00	49218.47	123411.84	2.51
3	380.00	47485.97	118537.44	2.50
4	370.00	45798.47	113753.03	2.48
5	360.00	44155.97	109058.63	2.47
6	350.00	42558.47	104454.22	2.45
7	340.00	41005.97	99939.81	2.44
8	330.00	39498.47	95515.41	2.42
9	320.00	38035.97	91181.00	2.40
10	310.00	36618.47	86936.59	2.37
11	300.00	35245.97	82782.19	2.35
12	290.00	33918.47	78717.78	2.32
13	280.00	32635.97	74743.38	2.29
14	270.00	31398.47	70858.97	2.26
15	260.00	30205.97	67064.56	2.22
16	250.00	29058.47	63360.16	2.18
17	240.00	27955.97	59745.75	2.14
18	230.00	26898.47	56221.34	2.09
19	220.00	25885.97	52786.94	2.04
20	210.00	24918.47	49442.53	1.98
21	200.00	23995.97	46188.13	1.92
22	190.00	23118.47	43023.72	1.86
23	180.00	22285.97	39949.31	1.79
24	170.00	21498.47	36964.91	1.72
25	160.00	20755.97	34070.50	1.64
26	150.00	20058.47	31266.09	1.56
27	140.00	19405.97	28551.69	1.47
28	130.00	18798.47	25927.28	1.38
29	120.00	18235.97	23392.88	1.28
30	110.00	17315.97	20563.47	1.19
31	100.00	16475.97	17894.06	1.09
left part < right part. Updating of iterations				
1	90.00	15715.97	15384.66	0.98
2	99.75	16456.00	17829.38	1.08
3	99.50	16436.07	17764.79	1.08
4	99.25	16416.20	17700.31	1.08
5	99.00	16396.37	17635.92	1.08
6	98.75	16376.60	17571.64	1.07
7	98.50	16356.87	17507.45	1.07

8	98.25	16337.20	17443.37	1.07
9	98.00	16317.57	17379.38	1.07
10	97.75	16298.00	17315.50	1.06
11	97.50	16278.47	17251.71	1.06
12	97.25	16259.00	17188.03	1.06
13	97.00	16239.57	17124.44	1.05
14	96.75	16220.20	17060.96	1.05
15	96.50	16200.87	16997.57	1.05
16	96.25	16181.60	16934.29	1.05
17	96.00	16162.37	16871.10	1.04
18	95.75	16143.20	16808.02	1.04
19	95.50	16124.07	16745.03	1.04
20	95.25	16105.00	16682.14	1.04
21	95.00	16085.97	16619.36	1.03
22	94.75	16067.00	16556.67	1.03
23	94.50	16048.07	16494.09	1.03
24	94.25	16029.20	16431.60	1.03
25	94.00	16010.37	16369.22	1.02
26	93.75	15991.60	16306.93	1.02
27	93.50	15972.87	16244.75	1.02
28	93.25	15954.20	16182.66	1.01
29	93.00	15935.57	16120.68	1.01
30	92.75	15917.00	16058.79	1.01
31	92.50	15898.47	15997.01	1.01
32	92.25	15880.00	15935.32	1.00
33	92.00	15861.57	15873.74	1.00
34	91.75	15843.20	15812.25	1.00

Final value of x is 91.75 mm and tensioning rebar Area is 402.12 mm2

3. Calculation of stress in tensionng zone of reinforcement

$$\sigma_{scr} = \frac{M^*}{A_s \cdot (d - x/3)} = \frac{0}{402.12 \cdot (400 - 91.75/3)} = 0.00 \text{ MPa}$$

4. Limiting stress check (8.6.1)

$$\text{For nominal bar diameter in tension zone } d_b = 16 \text{ mm the maximum stress is } 280.00 \text{ MPa (Table 8.6.1(A))}$$

STATUS OK!

$$\text{For centre-to-centre spacing in tension zone } 175.00 \text{ mm the maximum stress is } 260.00 \text{ MPa (Table 8.6.1(B))}$$

$$\sigma_{scr} = 0.00 \text{ MPa} \leq 280.00 \text{ MPa (For nominal bar diameter)}$$

$$\sigma_{scr} = 0.00 \text{ MPa} \leq 260.00 \text{ MPa (For centre-to-centre spacing)}$$

STATUS OK!

Shear check

8.2.7.1, 8.2.9, 8.2.6	SHEAR FORCE CAPACITY (Members with shear reinforcement) Section input data: Mean width of web $b_w = 450\ \mathrm{mm}$ Section concrete area $A_g = 244500\ \mathrm{mm}^2$ Tensioning rebar area $A_{st} = 1963.48\ \mathrm{mm}^2$ Working depth of reinforcement $d_0 = 380\ \mathrm{mm}$ Cross-sectional area of the shear reinforcement $A_{sv} = 157.08\ \mathrm{mm}^2$ Spacing of stirrups $s = 250.00\ \mathrm{mm}$ Given shear force $V^* = 4500\ \mathrm{kN}$ Corresponding axial force $N^* = 0\ \mathrm{kN}$ 1. Determine Concrete Shear Capacity (8.2.7.1) $$f'_{cv} = (f'_c)^{1/3} = (25)^{1/3} = 2.92\ \mathrm{MPa} \leq 4\ \mathrm{MPa}$$ $$\beta_1 = 1.1 \cdot (1.6 - \frac{d_0}{1000}) = 1.1 \cdot (1.6 - \frac{380}{1000}) = 1.34 \geq 1.1$$ $$\beta_2 = 1.0$$ $$\beta_3 = 1.0$$ $$V_{uc} = \beta_1 \cdot \beta_2 \cdot \beta_3 \cdot b_w \cdot d_0 \cdot f'_{cv} \cdot (\frac{A_{st}}{b_w \cdot d_0})^{1/3} = 1.34 \cdot 1.00 \cdot 1.00 \cdot 450 \cdot 380 \cdot 2.92 \cdot (\frac{1963.48}{450 \cdot 380})^{1/3} = 151.38\ \mathrm{kN}$$ 2. Determine required shear reinforcement (8.2.9, 8.2.6) $$V_{u,max} = 0.2 \cdot f'_c \cdot b_w \cdot d_0 = 0.2 \cdot 25 \cdot 450 \cdot 380 = 855.00\ \mathrm{kN}$$ $$V_{u,min} = V_{uc} + 0.1 \cdot \sqrt{f'_c} \cdot b_w \cdot d_0 = 151.38 + 0.1 \cdot \sqrt{25} \cdot 450 \cdot 380 = 85.65\ \mathrm{kN}$$ $$V_{u,min} < V_{uc} + 0.6 \cdot b_w \cdot d_0 = 151.38 + 0.6 \cdot 450 \cdot 380 = 102.75\ \mathrm{kN}$$ $$V_{u,min} = 102.75\ \mathrm{kN}$$ $$\phi = 0.75$$ $$A_{sv,min} = \frac{0.35 \cdot b_w \cdot s}{f_{sy,f}} = \frac{0.35 \cdot 450 \cdot 250}{500} = 78.75\ \mathrm{mm}^2$$ $$V_{us} = (A_{sv,min} \cdot f_{sy,f} \cdot d_0 / s) \cdot cot(\theta_v) = (78.75 \cdot 500 \cdot 380/250) \cdot cot(45.00) = 59.85\ \mathrm{kN}$$ 3. Design shear strength of a beam (8.2.2) $$V_u = V_{uc} + V_{us} = 151.38 + 59.85 = 211.23\ \mathrm{kN}$$ $$V^* = 4500.00\ \mathrm{kN} > V_u = 211.23\ \mathrm{kN}$$ $$A_{sv} = 157.08\ \mathrm{mm}^2 \geq A_{sv,min} = 78.75\ \mathrm{mm}^2$$	 **STATUS NG!** **STATUS OK!**