REFERENCES	CALCULATIONS	RESULTS

MEMBER #1 (SECTION POSITION 5000.0 mm) BEAM DESIGN REPORT

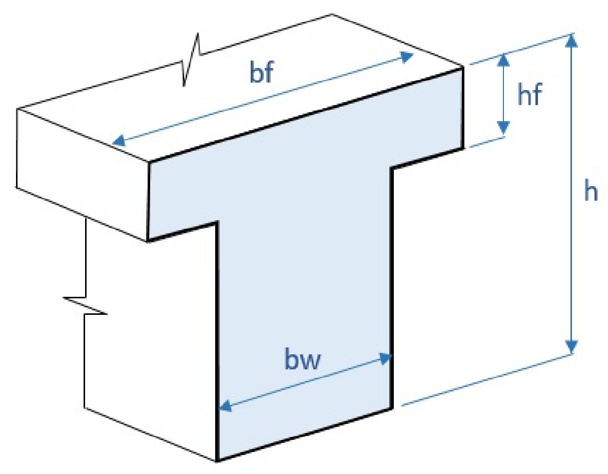
Code: AS 3600-2009

Project details

Project Name: Not Provided
Project ID: Not Provided
Company: Not Provided
Designer: Not Provided
Client: Not Provided
Project Notes: Not Provided
Project Units: Metric

General member design information

Dimentions:



Height $h = 600\ \mathrm{mm}$
Flange width $b_f = 1000\ \mathrm{mm}$
Flange thickness $h_f = 150\ \mathrm{mm}$
Web width $b_w = 400\ \mathrm{mm}$
Member length = $20000\ \mathrm{mm}$

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:
LC 1: 1.35DL (M = 168.75 kN-m, V = 33.75 kN)
LC 2: 1.2DL+1.5LL (M = 226.17 kN-m, V = 45.23 kN)
LC 3: 0.9DL+1.0WL (M = 112.50 kN-m, V = 22.50 kN)
LC 4: 1.2DL+1.0WL (M = 150.00 kN-m, V = 30.00 kN)
LC 5: 1.2DL+0.6LL+1.0WL (M = 180.47 kN-m, V = 36.09 kN)
LC 6: 1.0DL+1.0EL (M = 125.00 kN-m, V = 25.00 kN)
LC 7: 1.0DL+0.6LL+1.0EL (M = 155.47 kN-m, V = 31.09 kN)

Serviceabiity Limit State:
LC 1: 1.0DL (M = 125.00 kN-m)
LC 2: 1.0DL+1.0LL (M = 175.78 kN-m)

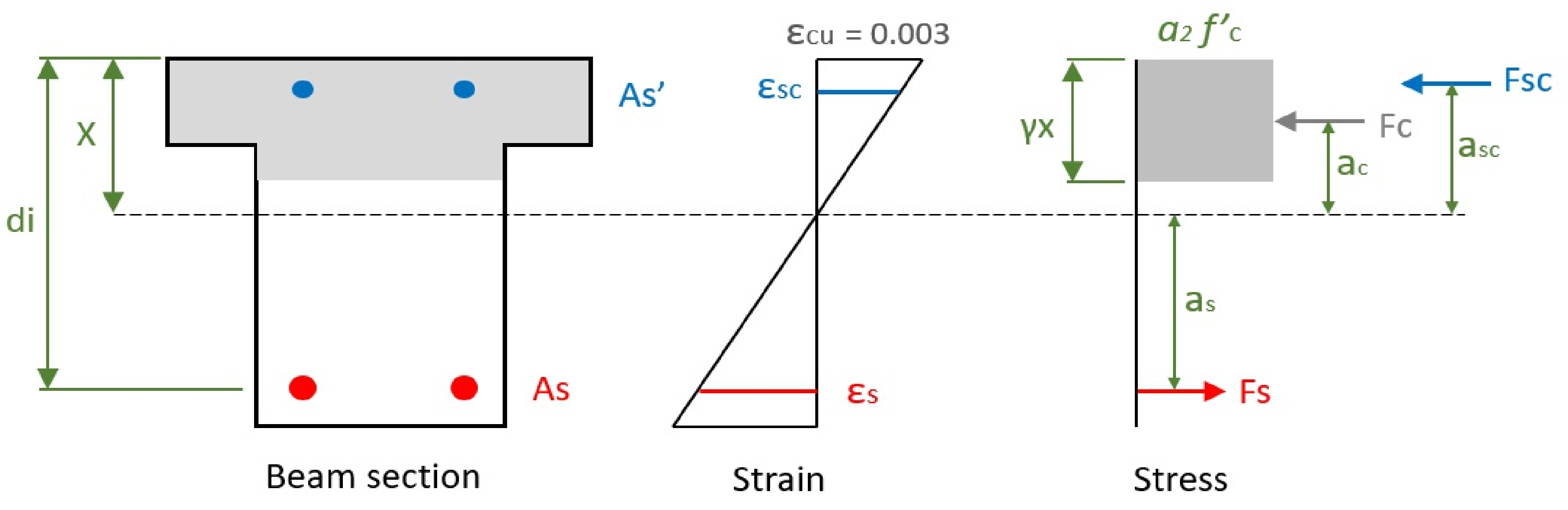
Accepted forces for section check:
Positive moment strength case : (M = 226.17 kN-m)
Positive moment service. case: (M = 175.78 kN-m)
Negative moment strength case: (M = 0.00 kN-m)
Negative moment service. case: (M = 0.00 kN-m)
Shear strength case: M = (226.17 kN-m, V = 45.23 kN)

DL - Dead Load
LL - Live Load
WL - Wind Load
LrL - Roof Live Load
RL - Rain Load
SL - Snow Load
EL - Earthquake Load

Flexure check (Positive bending moment case)

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

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 $d_0 = 540\ \mathrm{mm}$
Given bending moment $M^* = 226.17\ \mathrm{kN\text{-}m}$

Section Rebar

Depth di (mm)	bar diameter (mm)	bar area Asi (mm2)
540	25	490.87
540	25	490.87
540	25	490.87
540	25	490.87
50	16	201.06
50	16	201.06

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

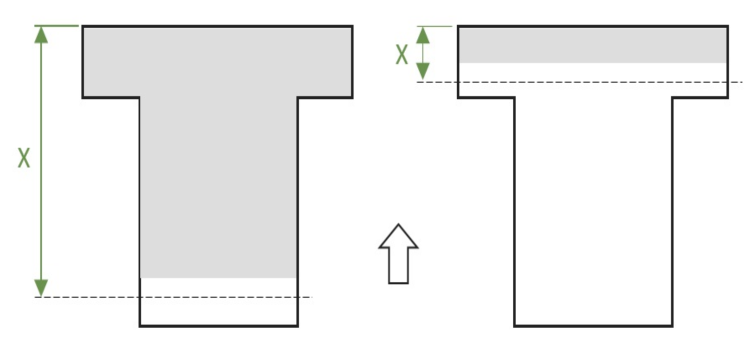
$$\alpha_2 = 1.0 - 0.003 \cdot \acute{f_c} = 1.0 - 0.003 \cdot 25 = 0.93$$

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

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

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

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 540 to 0 mm)

Iter.	x (mm)	kuo = x/do	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	540.0	1.00	5814.00	201.06	6015.06	0.00	Infinity
2	529.2	0.98	5735.97	201.06	5937.03	24.04	246.938
3	518.4	0.96	5657.94	201.06	5859.00	49.09	119.360
4	507.6	0.94	5579.91	201.06	5780.97	75.20	76.878
5	496.8	0.92	5501.88	201.06	5702.94	102.44	55.670
6	486.0	0.90	5423.85	201.06	5624.91	130.90	42.971

7	475.2	0.88	5345.82	201.06	5546.88	160.65	34.528
8	464.4	0.86	5267.79	201.06	5468.85	191.78	28.516
9	453.6	0.84	5189.76	201.06	5390.82	224.40	24.024
10	442.8	0.82	5111.73	201.06	5312.79	258.60	20.544
11	432.0	0.80	5033.70	201.06	5234.76	294.52	17.774
12	421.2	0.78	4955.67	201.06	5156.73	332.28	15.519
13	410.4	0.76	4877.64	201.06	5078.70	372.03	13.651
14	399.6	0.74	4799.61	201.06	5000.67	413.92	12.081
15	388.8	0.72	4721.58	201.06	4922.64	458.15	10.745
16	378.0	0.70	4643.55	201.06	4844.61	504.89	9.595
17	367.2	0.68	4565.52	201.06	4766.58	554.39	8.598
18	356.4	0.66	4487.49	201.06	4688.55	606.89	7.725
19	345.6	0.64	4409.46	201.06	4610.52	662.67	6.957
20	334.8	0.62	4331.43	201.06	4532.49	722.05	6.277
21	324.0	0.60	4253.40	201.06	4454.46	785.39	5.672
22	313.2	0.58	4175.37	201.06	4376.43	853.10	5.130
23	302.4	0.56	4097.34	201.06	4298.40	925.64	4.644
24	291.6	0.54	4019.31	199.90	4219.21	981.74	4.298
25	280.8	0.52	3941.28	198.31	4139.59	981.74	4.217
26	270.0	0.50	3863.25	196.59	4059.84	981.74	4.135
27	259.2	0.48	3785.22	194.73	3979.95	981.74	4.054
28	248.4	0.46	3707.19	192.71	3899.90	981.74	3.972
29	237.6	0.44	3629.16	190.50	3819.66	981.74	3.891
30	226.8	0.42	3551.13	188.08	3739.21	981.74	3.809
31	216.0	0.40	3473.10	185.42	3658.52	981.74	3.727
32	205.2	0.38	3395.07	182.48	3577.55	981.74	3.644
33	194.4	0.36	3317.04	179.22	3496.26	981.74	3.561
34	183.6	0.34	3239.01	175.57	3414.58	981.74	3.478
35	172.8	0.32	3121.20	171.46	3292.66	981.74	3.354
36	162.0	0.30	2926.12	166.81	3092.93	981.74	3.150
37	151.2	0.28	2731.05	161.49	2892.54	981.74	2.946
38	140.4	0.26	2535.97	155.35	2691.32	981.74	2.741
39	129.6	0.24	2340.90	148.19	2489.09	981.74	2.535
40	118.8	0.22	2145.82	139.73	2285.55	981.74	2.328
41	108.0	0.20	1950.75	129.57	2080.32	981.74	2.119
42	97.2	0.18	1755.67	117.16	1872.84	981.74	1.908
43	86.4	0.16	1560.60	101.65	1662.25	981.74	1.693
44	75.6	0.14	1365.52	81.70	1447.23	981.74	1.474
45	64.8	0.12	1170.45	55.11	1225.56	981.74	1.248
46	54.0	0.10	975.37	17.87	993.25	981.74	1.012
(Fc + Fcs) < Fs. Updating of iterations							
1	43.2	0.08	780.30	0.00	780.30	1019.72	0.765
2	53.8	0.10	971.47	16.97	988.45	981.74	1.007
3	53.6	0.10	967.57	16.07	983.64	981.74	1.002

4	53.4	0.10	963.67	15.16	978.83	981.74	0.997

Final value of x is 53.35 mm and flexural tension reinforcement area is 1963.48 mm2
Working depth of reinforcement $d = 540.00\ \mathrm{mm}$

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

$$\phi = 1.19 - 13 \cdot k_{uo}/12 = 1.19 - 13 \cdot 0.10/12 = 1.08$$

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

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

$$a = \gamma \cdot x = 0.85 \cdot 53.35 = 45.35\ \mathrm{mm} \leq a_{max} = \gamma \cdot k_u \cdot d_0 = 0.85 \cdot 0.36 \cdot 540 = 165.24\ \mathrm{mm}$$

2. Calculation moment resistance M_d (2.2.2)

$$\phi M_d = (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) \cdot \phi = (29.56 + 0.05 + 477.76) \cdot 0.80 = 405.90\ \text{kN-m}$$

$$M^* = 226.17\ \text{kN-m} \leq M_d = 405.90\ \text{kN-m}$$

STATUS OK!

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

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

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

$$\alpha_b = 0.2 + (\frac{b_f}{b_w} - 1) \cdot (0.25 \cdot (\frac{h_f}{h}) - 0.08) = 0.2 + (\frac{1000}{400} - 1) \cdot (0.25 \cdot (\frac{150}{600}) - 0.08) = 0.17$$

$$\alpha_b < 0.2 \cdot (\frac{b_f}{b_w})^{2/3} = 0.2 \cdot (\frac{1000}{400})^{2/3} = 0.37$$

$$\alpha_b = 0.37$$

$$A_{st,min} = \alpha_b \cdot (\frac{h}{d} \cdot \frac{f'_{ct,f}}{f_{sy}}) \cdot b_w \cdot d = 0.37 \cdot (\frac{600}{540.00}) \cdot \frac{3.00}{500} \cdot 400 \cdot 540.00 = 530.18\ \mathrm{mm}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$A_{st,max} = 0.04 \cdot b_w \cdot d = 0.04 \cdot 400 \cdot 540.00 = 8640.00\ \mathrm{mm}^2$$

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

$$A_{st} = 1963.48\ \mathrm{mm}^2 \leq A_{st,max} = 8640.00\ \mathrm{mm}^2$$

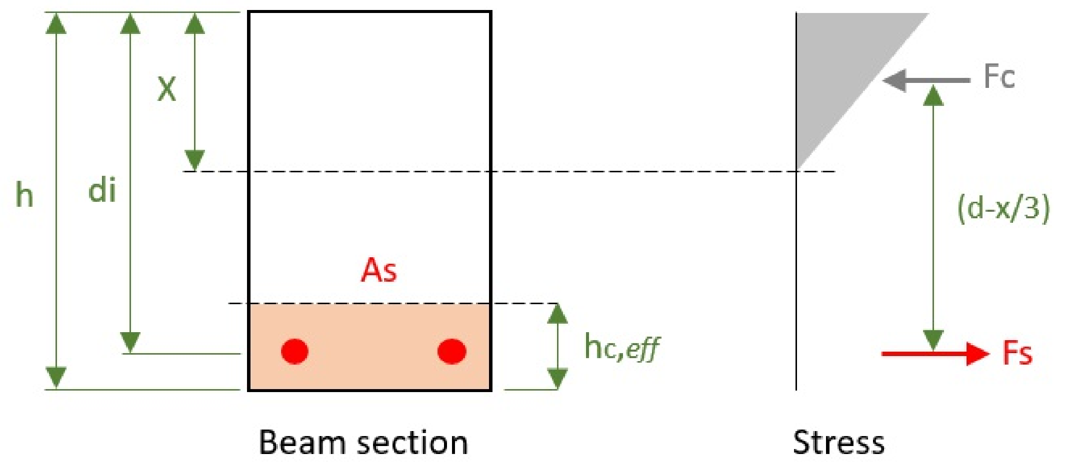
STATUS OK!

$$A_{st} = 1963.48\ \mathrm{mm}^2 \geq A_{st,min} = 530.18\ \mathrm{mm}^2$$

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 $A_g = 330000 \text{ mm}^2$
Web width $b_w = 400 \text{ mm}$
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 $M_s^* = 175.78 \text{ kN-m}$

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

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

Notional size of the member

$$t_h = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 330000.00}{3200} = 206.25 \text{ mm}$$

$$\alpha_2 = 1.0 + 1.12 \cdot e^{-0.008 \cdot t_h} = 1.0 + 1.12 \cdot e^{-0.008 \cdot 206.25} = 1.22$$

$$k_2 = \frac{\alpha_2 \cdot t^{0.8}}{t^{0.8} + 0.15 \cdot t_h} = \frac{1.22 \cdot 10000^{0.8}}{10000^{0.8} + 0.15 \cdot 206.25} = 1.19$$

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

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

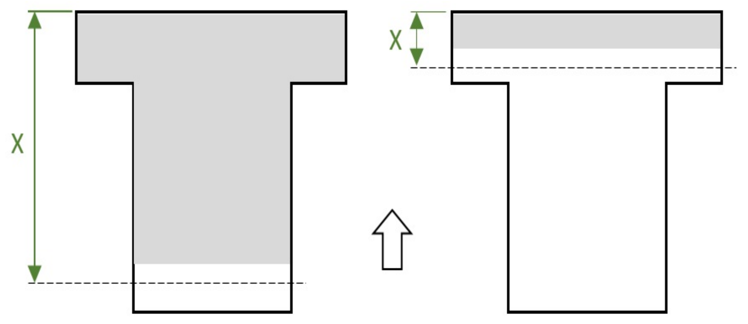
$$\phi_{cc} = k_2 \cdot k_3 \cdot k_4 \cdot k_5 \cdot \phi_{cc,b} = 1.19 \cdot 1.29 \cdot 0.70 \cdot 1.00 \cdot 4.20 = 4.51$$

Effective modulus

$$E_{eff} = \frac{E_{cm}}{1 + \phi_{cc}} = \frac{26700.00}{1 + 4.51} = 4847.41 \text{ MPa}$$

$$a_e = \frac{E_s}{E_{eff}} = \frac{200000}{4847.41} = 41.26$$

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 540 to 0 mm)

Iter.	x (mm)	As (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	540.00	0.00	109645.81	217945.47	1.988
2	529.20	1963.48	107336.33	211300.41	1.969
3	518.40	1963.48	105073.52	204748.67	1.949
4	507.60	1963.48	102857.36	198290.24	1.928
5	496.80	1963.48	100687.85	191925.12	1.906

6	486.00	1963.48	98565.01	185653.32	1.884
7	475.20	1963.48	96488.81	179474.83	1.860
8	464.40	1963.48	94459.28	173389.64	1.836
9	453.60	1963.48	92476.40	167397.78	1.810
10	442.80	1963.48	90540.17	161499.22	1.784
11	432.00	1963.48	88650.61	155693.97	1.756
12	421.20	1963.48	86807.69	149982.04	1.728
13	410.40	1963.48	85011.44	144363.42	1.698
14	399.60	1963.48	83261.84	138838.11	1.667
15	388.80	1963.48	81558.89	133406.11	1.636
16	378.00	1963.48	79902.61	128067.43	1.603
17	367.20	1963.48	78292.97	122822.05	1.569
18	356.40	1963.48	76730.00	117669.99	1.534
19	345.60	1963.48	75213.68	112611.24	1.497
20	334.80	1963.48	73744.01	107645.80	1.460
21	324.00	1963.48	72321.01	102773.68	1.421
22	313.20	1963.48	70944.65	97994.87	1.381
23	302.40	1963.48	69614.96	93309.36	1.340
24	291.60	1963.48	68331.92	88717.18	1.298
25	280.80	1963.48	67095.53	84218.30	1.255
26	270.00	1963.48	65905.81	79812.73	1.211
27	259.20	1963.48	64762.73	75500.48	1.166
28	248.40	1963.48	63666.32	71281.54	1.120
29	237.60	1963.48	62616.56	67155.91	1.072
30	226.80	1963.48	61613.45	63123.59	1.025
left part < right part. Updating of iterations					
1	216.00	1963.48	60657.01	59184.59	0.976
2	226.58	1963.48	61593.87	63043.90	1.024
3	226.37	1963.48	61574.30	62964.24	1.023
4	226.15	1963.48	61554.75	62884.62	1.022
5	225.94	1963.48	61535.22	62805.04	1.021
6	225.72	1963.48	61515.71	62725.49	1.020
7	225.50	1963.48	61496.22	62645.98	1.019
8	225.29	1963.48	61476.74	62566.51	1.018
9	225.07	1963.48	61457.29	62487.08	1.017
10	224.86	1963.48	61437.85	62407.68	1.016
11	224.64	1963.48	61418.43	62328.33	1.015
12	224.42	1963.48	61399.03	62249.00	1.014
13	224.21	1963.48	61379.65	62169.72	1.013
14	223.99	1963.48	61360.29	62090.47	1.012
15	223.78	1963.48	61340.95	62011.26	1.011
16	223.56	1963.48	61321.62	61932.09	1.010
17	223.34	1963.48	61302.32	61852.96	1.009
18	223.13	1963.48	61283.03	61773.86	1.008

19	222.91	1963.48	61263.76	61694.80	1.007
20	222.70	1963.48	61244.51	61615.78	1.006
21	222.48	1963.48	61225.28	61536.79	1.005
22	222.26	1963.48	61206.06	61457.84	1.004
23	222.05	1963.48	61186.87	61378.93	1.003
24	221.83	1963.48	61167.69	61300.06	1.002
25	221.62	1963.48	61148.54	61221.22	1.001
26	221.40	1963.48	61129.40	61142.43	1.000
27	221.18	1963.48	61110.28	61063.66	0.999

Final value of x is 221.18 mm and tensioning rebar area is 1963.48 mm2
Working depth of reinforcement $d = 540.00 \text{ mm}$

3. Calculation of stress in tensionng zone of reinforcement

$$\sigma_{scr} = \frac{M^*}{A_s \cdot (d - x/3)} = \frac{175780000}{1963.48 \cdot (540.00 - 221.18/3)} = 192.00 \text{ MPa}$$

4. Limiting stress check (8.6.1)

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

STATUS OK!

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

$$\sigma_{scr} = 192.00 \text{ MPa} \leq 203.75 \text{ MPa (For nominal bar diameter)}$$

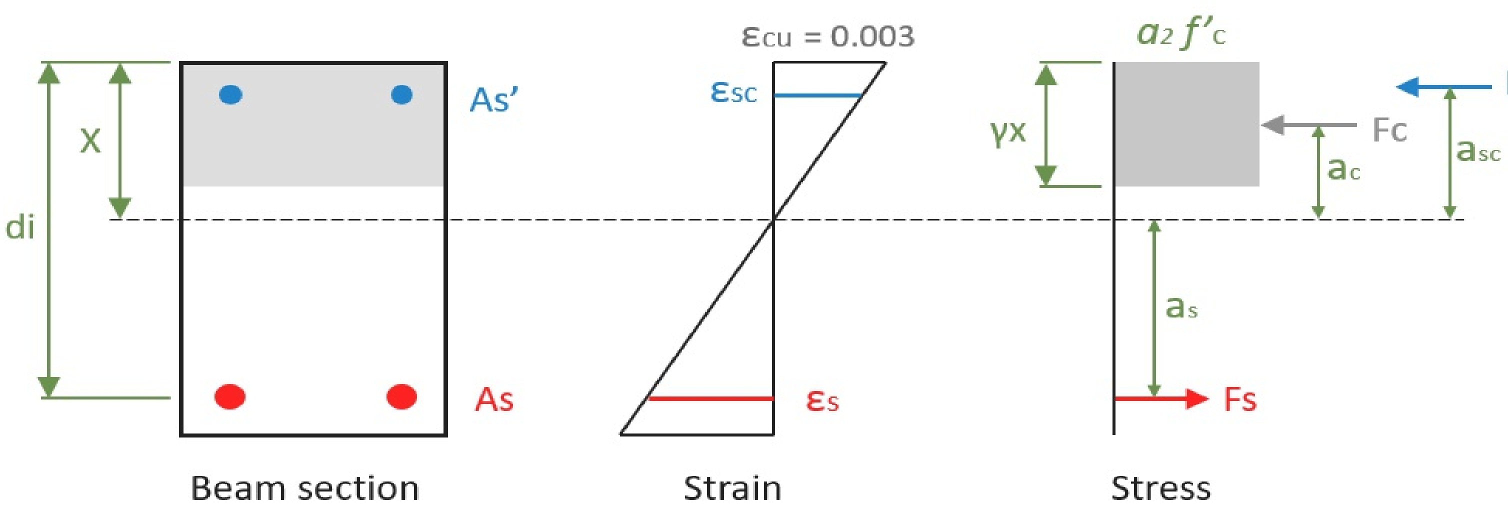
$$\sigma_{scr} = 192.00 \text{ MPa} \leq 340.00 \text{ MPa (For centre-to-centre spacing)}$$

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 $d_0 = 550 \text{ mm}$
Given bending moment $M^* = 0.00 \text{ kN-m}$

Section Rebar

Depth di (mm)	**bar diameter (mm)**	**bar area Asi (mm2)**
550	16	201.06
550	16	201.06
60	25	490.87
60	25	490.87
60	25	490.87
60	25	490.87

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

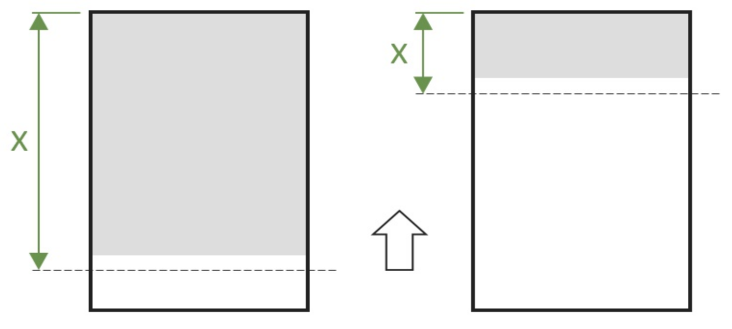
$$\alpha_2 = 1.0 - 0.003 \cdot \acute{f_c} = 1.0 - 0.003 \cdot 25 = 0.93$$

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

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

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

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 550 to 0 mm)

Iter.	x (mm)	kuo = x/do	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	550.0	1.00	5886.25	981.74	6867.99	0.00	Infinity
2	539.0	0.98	5806.77	981.74	6788.51	4.92	1378.681
3	528.0	0.96	5727.30	981.74	6709.04	10.05	667.367
4	517.0	0.94	5647.82	981.74	6629.56	15.40	430.482
5	506.0	0.92	5568.35	981.74	6550.09	20.98	312.204
6	495.0	0.90	5488.88	981.74	6470.61	26.81	241.369
7	484.0	0.88	5409.40	981.74	6391.14	32.90	194.255
8	473.0	0.86	5329.93	981.74	6311.66	39.28	160.697
9	462.0	0.84	5250.45	981.74	6232.19	45.96	135.610
10	451.0	0.82	5170.98	981.74	6152.72	52.96	116.172
11	440.0	0.80	5091.50	981.74	6073.24	60.32	100.687
12	429.0	0.78	5012.02	981.74	5993.76	68.05	88.077
13	418.0	0.76	4932.55	981.74	5914.29	76.19	77.624
14	407.0	0.74	4853.07	981.74	5834.81	84.77	68.830
15	396.0	0.72	4773.60	981.74	5755.34	93.83	61.339
16	385.0	0.70	4694.13	981.74	5675.86	103.40	54.891
17	374.0	0.68	4614.65	981.74	5596.39	113.54	49.290
18	363.0	0.66	4535.18	981.74	5516.91	124.29	44.387
19	352.0	0.64	4455.70	977.28	5432.98	135.72	40.032
20	341.0	0.62	4376.23	970.80	5347.02	147.88	36.159
21	330.0	0.60	4296.75	963.89	5260.64	160.85	32.706
22	319.0	0.58	4217.27	956.50	5173.78	174.71	29.613
23	308.0	0.56	4137.80	948.59	5086.39	189.57	26.831
24	297.0	0.54	4058.32	940.09	4998.42	201.06	24.860

25	286.0	0.52	3978.85	930.94	4909.79	201.06	24.420
26	275.0	0.50	3899.38	921.05	4820.43	201.06	23.975
27	264.0	0.48	3819.90	910.34	4730.24	201.06	23.527
28	253.0	0.46	3740.43	898.70	4639.12	201.06	23.073
29	242.0	0.44	3660.95	886.00	4546.95	201.06	22.615
30	231.0	0.42	3581.47	872.09	4453.57	201.06	22.150
31	220.0	0.40	3502.00	856.79	4358.79	201.06	21.679
32	209.0	0.38	3422.53	839.88	4262.41	201.06	21.200
33	198.0	0.36	3343.05	821.09	4164.14	201.06	20.711
34	187.0	0.34	3263.57	800.09	4063.67	201.06	20.211
35	176.0	0.32	3179.00	776.47	3955.47	201.06	19.673
36	165.0	0.30	2980.31	749.69	3730.00	201.06	18.552
37	154.0	0.28	2781.63	719.09	3500.72	201.06	17.411
38	143.0	0.26	2582.94	683.79	3266.72	201.06	16.248
39	132.0	0.24	2384.25	642.59	3026.84	201.06	15.054
40	121.0	0.22	2185.56	593.91	2779.47	201.06	13.824
41	110.0	0.20	1986.88	535.49	2522.37	201.06	12.545
42	99.0	0.18	1788.19	464.10	2252.28	201.06	11.202
43	88.0	0.16	1589.50	374.85	1964.35	201.06	9.770
44	77.0	0.14	1390.81	260.10	1650.91	201.06	8.211
45	66.0	0.12	1192.13	107.10	1299.22	201.06	6.462
46	55.0	0.10	993.44	0.00	993.44	308.16	3.224
47	44.0	0.08	794.75	0.00	794.75	629.46	1.263
(Fc + Fcs) < Fs. Updating of iterations							
1	33.0	0.06	596.06	0.00	596.06	1164.95	0.512
2	43.8	0.08	790.78	0.00	790.78	637.53	1.240
3	43.6	0.08	786.80	0.00	786.80	645.68	1.219
4	43.3	0.08	782.83	0.00	782.83	653.92	1.197
5	43.1	0.08	778.86	0.00	778.86	662.24	1.176
6	42.9	0.08	774.88	0.00	774.88	670.65	1.155
7	42.7	0.08	770.91	0.00	770.91	679.14	1.135
8	42.5	0.08	766.93	0.00	766.93	687.72	1.115
9	42.2	0.08	762.96	0.00	762.96	696.39	1.096
10	42.0	0.08	758.99	0.00	758.99	705.15	1.076
11	41.8	0.08	755.01	0.00	755.01	714.01	1.057
12	41.6	0.08	751.04	0.00	751.04	722.95	1.039
13	41.4	0.08	747.07	0.00	747.07	732.00	1.021
14	41.1	0.07	743.09	0.00	743.09	741.14	1.003
15	40.9	0.07	739.12	0.00	739.12	750.37	0.985

Final value of x is 40.92 mm and flexural tension reinforcement area is 2365.60 mm2
Working depth of reinforcement $d = 143.29\ \mathrm{mm}$

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

$$\phi = 1.19 - 13 \cdot k_{uo}/12 = 1.19 - 13 \cdot 0.07/12 = 1.11$$

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

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

$$a = \gamma \cdot x = 0.85 \cdot 40.92 = 34.78 \text{ mm} \leq a_{max} = \gamma \cdot k_u \cdot d_0 = 0.85 \cdot 0.36 \cdot 550 = 168.30 \text{ mm}$$

2. Calculation moment resistance M_d (2.2.2)

$$\phi M_d = (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) \cdot \phi = (17.39 + 0.00 + 112.84) \cdot 0.80 = 104.18 \text{ kN-m}$$

$$M^* = 0.00 \text{ kN-m} \leq M_d = 104.18 \text{ kN-m}$$

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

$$\alpha_b = 0.2 + (\frac{b_f}{b_w} - 1) \cdot (0.25 \cdot (\frac{h_f}{h}) - 0.08) = 0.2 + (\frac{1000}{400} - 1) \cdot (0.25 \cdot (\frac{150}{600}) - 0.08) = 0.17$$

$$\alpha_b < 0.2 \cdot (\frac{b_f}{b_w})^{2/3} = 0.2 \cdot (\frac{1000}{400})^{2/3} = 0.37$$

$$\alpha_b = 0.37$$

$$A_{st,min} = \alpha_b \cdot (\frac{h}{d} \cdot \frac{f'_{ct,f}}{f_{sy}}) \cdot b_w \cdot d = 0.37 \cdot (\frac{600}{143.29}) \cdot \frac{3.00}{500} \cdot 400 \cdot 143.29 = 530.18 \text{ mm}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$A_{st,max} = 0.04 \cdot b_w \cdot d = 0.04 \cdot 400 \cdot 143.29 = 2292.69 \text{ mm}^2$$

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

$$A_{st} = 2365.60 \text{ mm}^2 > A_{st,max} = 2292.69 \text{ mm}^2$$

STATUS NG!

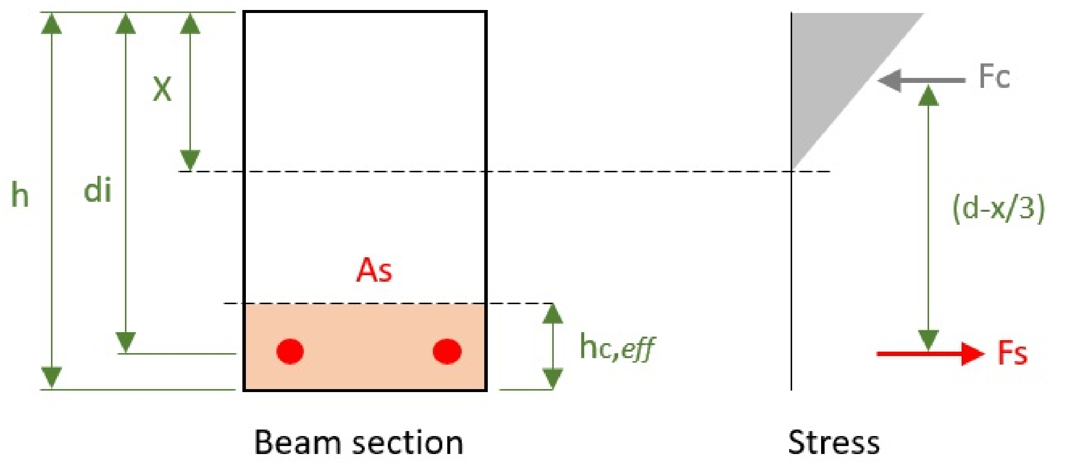
$$A_{st} = 2365.60 \text{ mm}^2 \geq A_{st,min} = 530.18 \text{ mm}^2$$

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 $A_g = 330000 \text{ mm}^2$
Web width $b_w = 400 \text{ mm}$
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 $E_c = 26700.00\ \text{MPa}$
Given bending moment $M_s^* = 0.00\ \text{kN-m}$

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

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

Notional size of the member

$$t_h = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 330000.00}{3200} = 206.25\ \text{mm}$$

$$\alpha_2 = 1.0 + 1.12 \cdot e^{-0.008 \cdot t_h} = 1.0 + 1.12 \cdot e^{-0.008 \cdot 206.25} = 1.22$$

$$k_2 = \frac{\alpha_2 \cdot t^{0.8}}{t^{0.8} + 0.15 \cdot t_h} = \frac{1.22 \cdot 10000^{0.8}}{10000^{0.8} + 0.15 \cdot 206.25} = 1.19$$

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

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

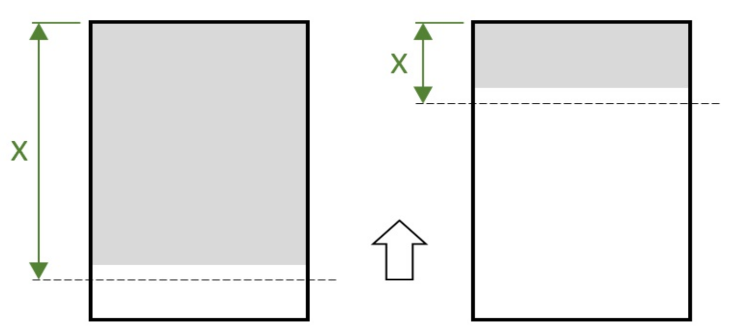
$$\phi_{cc} = k_2 \cdot k_3 \cdot k_4 \cdot k_5 \cdot \phi_{cc,b} = 1.19 \cdot 1.29 \cdot 0.70 \cdot 1.00 \cdot 4.20 = 4.51$$

Effective modulus

$$E_{eff} = \frac{E_{cm}}{1 + \phi_{cc}} = \frac{26700.00}{1 + 4.51} = 4847.41\ \text{MPa}$$

$$a_e = \frac{E_s}{E_{eff}} = \frac{200000}{4847.41} = 41.26$$

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 550 to 0 mm)

Iter.	x (mm)	As (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	550.00	0.00	81235.82	224181.49	2.760
2	539.00	402.12	78840.02	217326.26	2.757
3	528.00	402.12	76492.62	210567.83	2.753
4	517.00	402.12	74193.62	203906.20	2.748
5	506.00	402.12	71943.02	197341.37	2.743
6	495.00	402.12	69740.82	190873.34	2.737
7	484.00	402.12	67587.02	184502.11	2.730
8	473.00	402.12	65481.62	178227.68	2.722
9	462.00	402.12	63424.62	172050.05	2.713
10	451.00	402.12	61416.02	165969.22	2.702
11	440.00	402.12	59455.82	159985.19	2.691
12	429.00	402.12	57544.02	154097.96	2.678
13	418.00	402.12	55680.62	148307.53	2.664

14	407.00	402.12	53865.62	142613.90	2.648
15	396.00	402.12	52099.02	137017.08	2.630
16	385.00	402.12	50380.82	131517.05	2.610
17	374.00	402.12	48711.02	126113.82	2.589
18	363.00	402.12	47089.62	120807.39	2.565
19	352.00	402.12	45516.62	115597.76	2.540
20	341.00	402.12	43992.02	110484.93	2.511
21	330.00	402.12	42515.82	105468.90	2.481
22	319.00	402.12	41088.02	100549.67	2.447
23	308.00	402.12	39708.62	95727.24	2.411
24	297.00	402.12	38377.62	91001.61	2.371
25	286.00	402.12	37095.02	86372.78	2.328
26	275.00	402.12	35860.82	81840.75	2.282
27	264.00	402.12	34675.02	77405.52	2.232
28	253.00	402.12	33537.62	73067.09	2.179
29	242.00	402.12	32448.62	68825.46	2.121
30	231.00	402.12	31408.02	64680.63	2.059
31	220.00	402.12	30415.82	60632.60	1.993
32	209.00	402.12	29472.02	56681.37	1.923
33	198.00	402.12	28576.62	52826.94	1.849
34	187.00	402.12	27729.62	49069.31	1.770
35	176.00	402.12	26931.02	45408.48	1.686
36	165.00	402.12	26180.82	41844.45	1.598
37	154.00	402.12	25479.02	38377.22	1.506
38	143.00	402.12	24210.32	34406.19	1.421
39	132.00	402.12	22697.82	30307.56	1.335
40	121.00	402.12	21306.32	26450.93	1.241
41	110.00	402.12	20035.82	22836.30	1.140
42	99.00	402.12	18886.32	19463.67	1.031
left part < right part. Updating of iterations					
1	88.00	402.12	17857.82	16333.04	0.915
2	98.78	402.12	18864.57	19398.68	1.028
3	98.56	402.12	18842.86	19333.80	1.026
4	98.34	402.12	18821.20	19269.01	1.024
5	98.12	402.12	18799.59	19204.31	1.022
6	97.90	402.12	18778.03	19139.72	1.019
7	97.68	402.12	18756.51	19075.22	1.017
8	97.46	402.12	18735.05	19010.81	1.015
9	97.24	402.12	18713.63	18946.51	1.012
10	97.02	402.12	18692.26	18882.30	1.010
11	96.80	402.12	18670.94	18818.18	1.008
12	96.58	402.12	18649.67	18754.17	1.006
13	96.36	402.12	18628.45	18690.25	1.003
14	96.14	402.12	18607.27	18626.42	1.001

15	95.92	402.12	18586.15	18562.70	0.999

Final value of x is 95.92 mm and tensioning rebar area is 402.12 mm2
Working depth of reinforcement $d = 550.00\ \mathrm{mm}$

3. Calculation of stress in tensionng zone of reinforcement

$$\sigma_{scr} = \frac{M^*}{A_s \cdot (d - x/3)} = \frac{0}{402.12 \cdot (550.00 - 95.92/3)} = 0.00\ \mathrm{MPa}$$

4. Limiting stress check (8.6.1)

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

$$\text{For centre-to-centre spacing in tension zone } 150.00\ \mathrm{mm} \text{ the maximum stress is } 280.00\ \mathrm{MPa}\ (\text{Table } 8.6.1(\mathrm{B}))$$

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

STATUS OK!

$$\sigma_{scr} = 0.00\ \mathrm{MPa} \leq 280.00\ \mathrm{MPa}\ (\text{For centre-to-centre spacing})$$

STATUS OK!

8.2.7.1, 8.2.9, 8.2.6

Shear check

SHEAR FORCE CAPACITY (Members with shear reinforcement)

Section input data:
Mean width of web $b_w = 400\ \mathrm{mm}$
Section concrete area $A_g = 330000\ \mathrm{mm}^2$
Tensioning rebar area $A_{st} = 1963.48\ \mathrm{mm}^2$
Working depth of reinforcement $d_0 = 540\ \mathrm{mm}$
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 $V^* = 45.23\ \mathrm{kN}$
Corresponding axial force $N^* = 0.00\ \mathrm{kN}$

1. Determine Concrete Shear Capacity (8.2.7.1)

$$f'_{cv} = (f'_c)^{1/3} = (25)^{1/3} = 2.92\ \mathrm{MPa} \leq 4\ \mathrm{MPa}$$

$$\beta_1 = 1.1 \cdot (1.6 - \frac{d_0}{1000}) = 1.1 \cdot (1.6 - \frac{540}{1000}) = 1.17 \geq 1.1$$

$$\beta_2 = 1.0$$

$$\beta_3 = 1.0$$

$$V_{uc} = \beta_1 \cdot \beta_2 \cdot \beta_3 \cdot b_w \cdot d_0 \cdot f'_{cv} \cdot (\frac{A_{st}}{b_w \cdot d_0})^{1/3} = 1.17 \cdot 1.00 \cdot 1.00 \cdot 400 \cdot 540 \cdot 2.92 \cdot (\frac{1963.48}{400 \cdot 540})^{1/3} = 153.69\ \mathrm{kN}$$

2. Determine required shear reinforcement (8.2.9, 8.2.6)

$$V_{u,max} = 0.2 \cdot f'_c \cdot b_w \cdot d_0 = 0.2 \cdot 25 \cdot 400 \cdot 540 = 1080.00\ \mathrm{kN}$$

$$V_{u,min} = V_{uc} + 0.1 \cdot \sqrt{f'_c} \cdot b_w \cdot d_0 = 153.69 + 0.1 \cdot \sqrt{25} \cdot 400 \cdot 540 = 108.15\ \mathrm{kN}$$

$$V_{u,min} < V_{uc} + 0.6 \cdot b_w \cdot d_0 = 153.69 + 0.6 \cdot 400 \cdot 540 = 129.75\ \mathrm{kN}$$

$$V_{u,min} = 129.75\ \mathrm{kN}$$

$$\phi = 0.75$$

$$A_{sv,min} = \frac{0.35 \cdot b_w \cdot s}{f_{sy,f}} = \frac{0.35 \cdot 400 \cdot 250}{500} = 70.00\ \mathrm{mm}^2$$

$$V_{us} = (A_{sv,min} \cdot f_{sy,f} \cdot d_0/s) \cdot cot(\theta_v) = (70.00 \cdot 500 \cdot 540/250) \cdot cot(30.00) = 130.94 \text{ kN}$$

3. Design shear strength of a beam (8.2.2)

$$V_u = V_{uc} + V_{us} = 153.69 + 130.94 = 284.64 \text{ kN}$$

$$V^* = 45.23 \text{ kN} \leq V_u = 284.64 \text{ kN}$$

STATUS OK!

$$A_{sv} = 157.08 \text{ mm}^2 \geq A_{sv,min} = 70.00 \text{ mm}^2$$

STATUS OK!

8.5.3.1, 8.5.3.2

Deflection check

DEFLECTION OF BEAM (short-term)

Section input data:

Characteristic flexural tensile strength of concrete $f'_{ct,f} = 0.6 \cdot \sqrt{f'_c} = 3.00 \text{ MPa}$
Uniaxial tensile strength $f_{ct} = 0.6 \cdot f_{ct,f} = 1.80 \text{ MPa}$
Given service bending moment due to Dead Load $M^*_{s,DL} = 125.00 \text{ kN-m}$
Given service bending moment due to Lead Load $M^*_{s,LL} = 50.78 \text{ kN-m}$
Typical dflection factor $k = 0.104$
Deflection limitation $L/250$
Short term factor $\psi_S = 0.7$
Short term factor $\psi_L = 0.4$
Member span $L = 10000 \text{ mm}$
Modulus of Elasticity $E_c = 26700.00 \text{ MPa}$
Modulus of Elasticity $E_s = 200000.00 \text{ MPa}$
Modular ratio $a_e = E_s/E_c 7.49$

Searching of neutral axis x (from 540 to 0 mm)

Iter.	x (mm)	As (mm2)	Asc (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	540.00	0.00	402.12	73162.77	174808.72	2.389
2	529.20	1963.48	402.12	70853.30	169026.40	2.386
3	518.40	1963.48	402.12	68590.49	163337.39	2.381
4	507.60	1963.48	402.12	66374.33	157741.70	2.377
5	496.80	1963.48	402.12	64204.82	152239.32	2.371
6	486.00	1963.48	402.12	62081.97	146830.25	2.365
7	475.20	1963.48	402.12	60005.78	141514.49	2.358
8	464.40	1963.48	402.12	57976.25	136292.04	2.351
9	453.60	1963.48	402.12	55993.37	131162.91	2.342
10	442.80	1963.48	402.12	54057.14	126127.09	2.333
11	432.00	1963.48	402.12	52167.57	121184.58	2.323
12	421.20	1963.48	402.12	50324.66	116335.38	2.312
13	410.40	1963.48	402.12	48528.41	111579.49	2.299
14	399.60	1963.48	402.12	46778.81	106916.92	2.286
15	388.80	1963.48	402.12	45075.86	102347.65	2.271
16	378.00	1963.48	402.12	43419.57	97871.70	2.254
17	367.20	1963.48	402.12	41809.94	93489.06	2.236
18	356.40	1963.48	402.12	40246.97	89199.74	2.216
19	345.60	1963.48	402.12	38730.65	85003.72	2.195
20	334.80	1963.48	402.12	37260.98	80901.02	2.171
21	324.00	1963.48	402.12	35837.97	76891.63	2.146

22	313.20	1963.48	402.12	34461.62	72975.55	2.118
23	302.40	1963.48	402.12	33131.93	69152.79	2.087
24	291.60	1963.48	402.12	31848.89	65423.33	2.054
25	280.80	1963.48	402.12	30612.50	61787.19	2.018
26	270.00	1963.48	402.12	29422.77	58244.36	1.980
27	259.20	1963.48	402.12	28279.70	54794.84	1.938
28	248.40	1963.48	402.12	27183.29	51438.63	1.892
29	237.60	1963.48	402.12	26133.53	48175.74	1.843
30	226.80	1963.48	402.12	25130.42	45006.16	1.791
31	216.00	1963.48	402.12	24173.97	41929.89	1.735
32	205.20	1963.48	402.12	23264.18	38946.93	1.674
33	194.40	1963.48	402.12	22401.05	36057.28	1.610
34	183.60	1963.48	402.12	21584.57	33260.95	1.541
35	172.80	1963.48	402.12	20814.74	30557.93	1.468
36	162.00	1963.48	402.12	20091.57	27948.22	1.391
37	151.20	1963.48	402.12	19415.06	25431.82	1.310
38	140.40	1963.48	402.12	17948.85	22200.03	1.237
39	129.60	1963.48	402.12	16490.85	19092.65	1.158
40	118.80	1963.48	402.12	15149.49	16218.56	1.071
left part < right part. Updating of iterations						
1	108.00	1963.48	402.12	13924.77	13577.74	0.975
2	118.58	1963.48	402.12	15123.86	16163.46	1.069
3	118.37	1963.48	402.12	15098.26	16108.45	1.067
4	118.15	1963.48	402.12	15072.72	16053.53	1.065
5	117.94	1963.48	402.12	15047.22	15998.71	1.063
6	117.72	1963.48	402.12	15021.77	15943.98	1.061
7	117.50	1963.48	402.12	14996.37	15889.34	1.060
8	117.29	1963.48	402.12	14971.01	15834.80	1.058
9	117.07	1963.48	402.12	14945.70	15780.35	1.056
10	116.86	1963.48	402.12	14920.44	15726.00	1.054
11	116.64	1963.48	402.12	14895.22	15671.73	1.052
12	116.42	1963.48	402.12	14870.05	15617.56	1.050
13	116.21	1963.48	402.12	14844.92	15563.49	1.048
14	115.99	1963.48	402.12	14819.85	15509.50	1.047
15	115.78	1963.48	402.12	14794.81	15455.62	1.045
16	115.56	1963.48	402.12	14769.83	15401.82	1.043
17	115.34	1963.48	402.12	14744.89	15348.12	1.041
18	115.13	1963.48	402.12	14720.00	15294.51	1.039
19	114.91	1963.48	402.12	14695.16	15240.99	1.037
20	114.70	1963.48	402.12	14670.36	15187.57	1.035
21	114.48	1963.48	402.12	14645.61	15134.24	1.033
22	114.26	1963.48	402.12	14620.90	15081.00	1.031
23	114.05	1963.48	402.12	14596.25	15027.86	1.030
24	113.83	1963.48	402.12	14571.64	14974.81	1.028

25	113.62	1963.48	402.12	14547.07	14921.85	1.026
26	113.40	1963.48	402.12	14522.55	14868.99	1.024
27	113.18	1963.48	402.12	14498.08	14816.22	1.022
28	112.97	1963.48	402.12	14473.66	14763.55	1.020
29	112.75	1963.48	402.12	14449.28	14710.96	1.018
30	112.54	1963.48	402.12	14424.95	14658.47	1.016
31	112.32	1963.48	402.12	14400.66	14606.08	1.014
32	112.10	1963.48	402.12	14376.43	14553.77	1.012
33	111.89	1963.48	402.12	14352.24	14501.56	1.010
34	111.67	1963.48	402.12	14328.09	14449.45	1.008
35	111.46	1963.48	402.12	14303.99	14397.42	1.007
36	111.24	1963.48	402.12	14279.94	14345.49	1.005
37	111.02	1963.48	402.12	14255.94	14293.66	1.003
38	110.81	1963.48	402.12	14231.98	14241.91	1.001
39	110.59	1963.48	402.12	14208.07	14190.26	1.00

Value of x is 110.59 mm
Tensioning rebar area $A_s = 1963.48\ \mathrm{mm}^2$
Compression rebar area $A_{sc} = 402.12\ \mathrm{mm}^2$
Working depth of reinforcement $d = 540.00\ \mathrm{mm}$

1. Calculate the moment of inertia of uncracked section I_{uc}

$$I_{uc} = 1/3(h \cdot b_w^3 + b_f \cdot h_f^3) = 1/3(600 \cdot 400^3 + 1000 \cdot 150^3) = 13925000000.00\ \mathrm{mm}^4$$

2. Calculate the moment of inertia of cracked section I_{cr}

$$I_{cr} = \frac{b \cdot x^3}{3} + a_e \cdot A_s \cdot (d - x)^2 = \frac{400 \cdot 110.59^3}{3} + 7.49 \cdot 1963.48 \cdot (540.00 - 110.59)^2 = 2892321126.52\ \mathrm{mm}^4$$

3. Moment that will cause cracking of the section:

$$M_{cr} = \frac{f_{ct} \cdot I_{uc}}{y_t} = \frac{1.80 \cdot 13925000000.00}{361.36} = 69.36\ \text{kN-m}$$

4. Calculate effective second movement of area (8.5.3.1)

$$\rho = \frac{A_s}{b \cdot d} = \frac{1963.48}{400 \cdot 540.00} = 0.009090 \geq 0.005 \rightarrow I_{ef,max} = I_{uc} = 13925000000.00\ \mathrm{mm}^4$$

$$I_{ef} = I_{cr} + (I_{uc} - I_{cr}) \cdot (M_{cr}/(M^*_{s,DL} + M^*_{s,LL}))^3 = 2892321126.52 + (13925000000.00 - 2892321126.52) \cdot (69.36/(125.00 + 50.78)^3 = 3570184830.49\ \mathrm{mm}^4$$

$$I_{ef} \leq I_{ef,max} \rightarrow I_{ef} = 3570184830.49\ \mathrm{mm}^4$$

5. Calculate the curvature of section

$$\text{Immediate Dead Load: } (1/r)_{ef,DL} = \frac{M^*_{s,DL}}{E_c \cdot I_{ef}} = \frac{125.00}{26700.00 \cdot 3570184830.49} = 0.00000131\ /\mathrm{mm}$$

$$\text{Immediate Live Load: } (1/r)_{ef,LL} = \frac{M^*_{s,LL}}{E_c \cdot I_{ef}} = \frac{50.78}{26700.00 \cdot 3570184830.49} = 0.00000053\ /\mathrm{mm}$$

6. Calculate long term deflection factor (8.5.3.2)

$$k_{cs} = 2 - 1.2 \cdot (\frac{A_{sc}}{A_s}) = 2 - 1.2 \cdot (\frac{402.12}{1963.48}) = 1.75$$

$$k_{cs} \geq 0.8$$

7. Calculate deflections based on section curvature

	Immediate Dead Load: $\triangle_{DL} = k \cdot L^2 \cdot (1/r)_{ef,DL} = 0.104 \cdot 10000^2 \cdot 0.00000131 = 13.6377 \text{ mm}$ Immediate Live Load: $\triangle_{LL} = k \cdot L^2 \cdot (1/r)_{ef,LL} = 0.104 \cdot 10000^2 \cdot 0.00000053 = 5.5402 \text{ mm}$ Short term: $\triangle_s = \triangle_{DL} + \psi_s \cdot \triangle_{LL} = 13.64 + 0.70 \cdot 5.5402 = 17.52 \text{ mm}$ Sustained position: $\triangle_{sus} = \triangle_{DL} + \psi_L \cdot \triangle_{LL} = 13.64 + 0.40 \cdot 5.5402 = 15.85 \text{ mm}$ Total: $\triangle_{total} = \triangle_s + k_{cs} \cdot \triangle_{sus} = 17.52 + 1.75 \cdot 15.85 = 45.3272 \text{ mm}$ $\triangle_{total} = 45.3272 \text{ mm} > \frac{L}{250} = \frac{10000}{250} = 40.00 \text{ mm}$	**STATUS NG!**