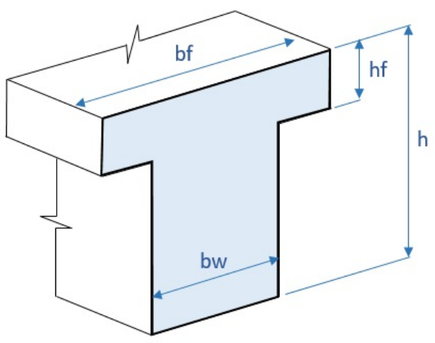
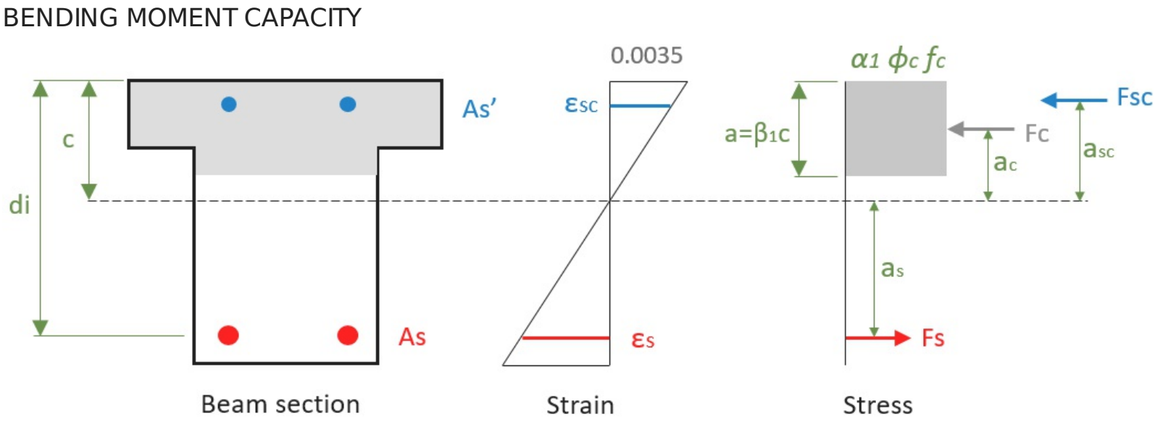
REFERENCES	CALCULATIONS	RESULTS
Code: CSA A23.3-14	**MEMBER #1 (SECTION POSITION 0.0 mm) BEAM DESIGN REPORT** **Project details** **Project Name:** **Project ID:** Company: Designer: Client: Project Notes: Project Units: Metric **General member design information** Dimentions:  Height $h = 700\ \mathrm{mm}$ Flange width $b_f = 1100\ \mathrm{mm}$ Flange thickness $h_f = 100\ \mathrm{mm}$ Web width $b_w = 300\ \mathrm{mm}$ Member length = $5000\ \mathrm{mm}$ Material properties: Concrete strength $f_c = 25\ \mathrm{MPa}$ Steel strength of longitudinal rebar $f_y = 400\ \mathrm{MPa}$ Steel strength of shear rebar $f_{yt} = 400\ \mathrm{MPa}$ Limit crack control parameter $z_{lim} = 30000\ \mathrm{N/mm}$ **Load Combinations (Ultimate Limit State)** For axial force in section: LC1: USER = $0\ \mathrm{kN}$ For bending moment in section: LC1: USER = $0\ \mathrm{kN\text{-}m}$ For shear force in section: LC1: USER = $0\ \mathrm{kN}$ **Load Combinations (Serviceability Limit State)** For bending moment in section: LC1: USER = $0\ \mathrm{kN\text{-}m}$	
8.4, 10.1, 10.5	**Flexure check (Positive bending moment case)** BENDING MOMENT CAPACITY  Section input data: Ultimate strain in concrete $e_{cmax} = 0.0035$ Distance to the outermost layer of tensile reinforcement $d = 650\ \mathrm{mm}$ Given bending moment $M = 0.00\ \mathrm{kN\text{-}m}$ Concrete resistance factor (8.4.2) $\phi_c = 0.65$ Reinforcement resistance factor (8.4.3) $\phi_s = 0.85$ Design yield strain of rebar $e_y = f_s/E_s = 400/200000 = 0.00200$ Section Rebar	

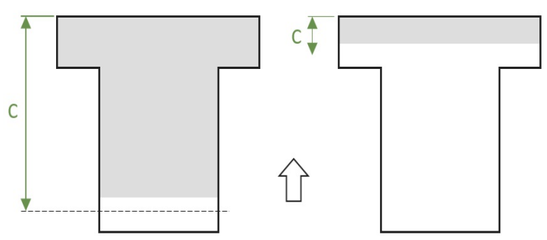
Depth di (mm)	bar diameter (mm)	bar area Asi (mm^2)
650	19.55	300.18
650	19.55	300.18
650	19.55	300.18

Rectangular compression block factors (10.1.7)

$$\alpha_1 = 0.85 - 0.0015 \cdot f_c = 0.85 - 0.0015 \cdot 25 = 0.81$$

$$\beta_1 = 0.97 - 0.0025 \cdot f_c = 0.97 - 0.0025 \cdot 25 = 0.91$$

1. Calculation of neutral axis depth c



Calculation is based on iterative process:
- Assume c
- Calculate concrete force $F_c = \alpha_1 \cdot \phi_c \cdot f_c \cdot \int_{dA} \cdot \beta_1 \cdot c$
- Calculate compression force in steel $F_{cs} = \phi_s \cdot \sum A_{s,i}^{'} \cdot f_{s,i}$
- Calculate tensioning force in steel $F_s = \phi_s \cdot \sum A_{s,i} \cdot f_{s,i}$
- 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 $e_s = e_{cu} \cdot (c - d)/c$
Reinforcement strains below axis $e_s = e_{cu} \cdot (d - c)/c$

Searching of neutral axis c (from 650 to 0 mm)

Iter.	c (mm)	a (mm)	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	650.0	589.9	3392.71	0.00	3392.71	0.00	Infinity
2	637.0	578.1	3345.98	0.00	3345.98	10.94	305.984
3	624.0	566.3	3299.25	0.00	3299.25	22.33	147.777
4	611.0	554.5	3252.52	0.00	3252.52	34.20	95.099
5	598.0	542.7	3205.79	0.00	3205.79	46.59	68.804
6	585.0	530.9	3159.06	0.00	3159.06	59.54	53.062
7	572.0	519.1	3112.33	0.00	3112.33	73.07	42.596
8	559.0	507.3	3065.60	0.00	3065.60	87.23	35.145
9	546.0	495.5	3018.87	0.00	3018.87	102.06	29.579
10	533.0	483.7	2972.15	0.00	2972.15	117.62	25.269
11	520.0	471.9	2925.42	0.00	2925.42	133.96	21.839
12	507.0	460.1	2878.69	0.00	2878.69	151.13	19.048
13	494.0	448.3	2831.96	0.00	2831.96	169.21	16.737
14	481.0	436.5	2785.23	0.00	2785.23	188.26	14.794
15	468.0	424.7	2738.50	0.00	2738.50	208.37	13.142
16	455.0	412.9	2691.77	0.00	2691.77	229.64	11.722
17	442.0	401.1	2645.04	0.00	2645.04	252.15	10.490
18	429.0	389.3	2598.31	0.00	2598.31	276.03	9.413
19	416.0	377.5	2551.58	0.00	2551.58	301.40	8.466
20	403.0	365.7	2504.85	0.00	2504.85	306.18	8.181
21	390.0	353.9	2458.12	0.00	2458.12	306.18	8.028
22	377.0	342.1	2411.40	0.00	2411.40	306.18	7.876

23	364.0	330.3	2364.67	0.00	2364.67	306.18	7.723
24	351.0	318.5	2317.94	0.00	2317.94	306.18	7.570
25	338.0	306.7	2271.21	0.00	2271.21	306.18	7.418
26	325.0	294.9	2224.48	0.00	2224.48	306.18	7.265
27	312.0	283.1	2177.75	0.00	2177.75	306.18	7.113
28	299.0	271.3	2131.02	0.00	2131.02	306.18	6.960
29	286.0	259.5	2084.29	0.00	2084.29	306.18	6.807
30	273.0	247.7	2037.56	0.00	2037.56	306.18	6.655
31	260.0	235.9	1990.83	0.00	1990.83	306.18	6.502
32	247.0	224.2	1944.10	0.00	1944.10	306.18	6.349
33	234.0	212.4	1897.37	0.00	1897.37	306.18	6.197
34	221.0	200.6	1850.65	0.00	1850.65	306.18	6.044
35	208.0	188.8	1803.92	0.00	1803.92	306.18	5.892
36	195.0	177.0	1757.19	0.00	1757.19	306.18	5.739
37	182.0	165.2	1710.46	0.00	1710.46	306.18	5.586
38	169.0	153.4	1663.73	0.00	1663.73	306.18	5.434
39	156.0	141.6	1617.00	0.00	1617.00	306.18	5.281
40	143.0	129.8	1570.27	0.00	1570.27	306.18	5.129
41	130.0	118.0	1523.54	0.00	1523.54	306.18	4.976
42	117.0	106.2	1476.81	0.00	1476.81	306.18	4.823
43	104.0	94.4	1370.72	0.00	1370.72	306.18	4.477
44	91.0	82.6	1199.38	0.00	1199.38	306.18	3.917
45	78.0	70.8	1028.04	0.00	1028.04	306.18	3.358
46	65.0	59.0	856.70	0.00	856.70	306.18	2.798
47	52.0	47.2	685.36	0.00	685.36	306.18	2.238
48	39.0	35.4	514.02	0.00	514.02	306.18	1.679
49	26.0	23.6	342.68	0.00	342.68	306.18	1.119
(Fc + Fcs) < Fs. Updating of iterations							
1	13.0	11.8	171.34	0.00	171.34	306.18	0.560
2	25.7	23.4	339.25	0.00	339.25	306.18	1.108
3	25.5	23.1	335.83	0.00	335.83	306.18	1.097
4	25.2	22.9	332.40	0.00	332.40	306.18	1.086
5	25.0	22.7	328.97	0.00	328.97	306.18	1.074
6	24.7	22.4	325.55	0.00	325.55	306.18	1.063
7	24.4	22.2	322.12	0.00	322.12	306.18	1.052
8	24.2	21.9	318.69	0.00	318.69	306.18	1.041
9	23.9	21.7	315.27	0.00	315.27	306.18	1.030
10	23.7	21.5	311.84	0.00	311.84	306.18	1.018
11	23.4	21.2	308.41	0.00	308.41	306.18	1.007
12	23.1	21.0	304.99	0.00	304.99	306.18	0.996

Final value of c is 23.14 mm, flexural tension reinforcement area is 900.54 mm^2 and flexural compression reinforcement area is 0.00 mm^2
Working depth of reinforcement $d = 650.00\ \text{mm}$

2. Calculation of moment resistance M_r

$$M_r = F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s = 3.86 + 0.00 + 191.93 = 195.79\ \text{kN-m}$$

$$M = 0.00 \text{ kN-m} \leq M_r = 195.79 \text{ kN-m (Ratio: 0.000)}$$

STATUS OK!
Ratio: 0.000

3. Minimum required flexural tension reinforcement in a beam section (10.5.1.2)

Width of tension zone $b_t = 450.00 \text{ mm} (b_f \text{ exeeds } 1.5 b_w)$

$$A_{st,min} = \frac{0.2 \cdot \sqrt{f_c}}{f_y} \cdot b_t \cdot h = \frac{0.2 \cdot \sqrt{25}}{400} \cdot 450 \cdot 700 = 787.50 \text{ mm}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$A_{st,max} = 0.04 \cdot b_w \cdot d = 0.04 \cdot 300 \cdot 650.00 = 7800.00 \text{ mm}^2$$

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

$$A_{st} = 900.54 \text{ mm}^2 \leq A_{st,max} = 7800.00 \text{ mm}^2 \text{ (Ratio: 0.115)}$$

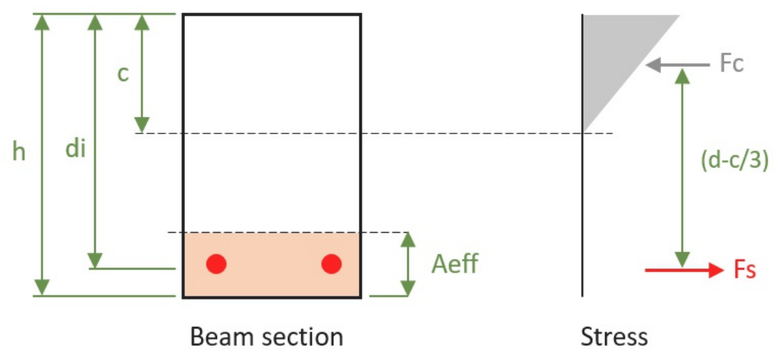
STATUS OK!
Ratio: 0.115

$$A_{st} = 900.54 \text{ mm}^2 \geq A_{st,min} = 787.50 \text{ mm}^2 \text{ (Ratio: 0.874)}$$

STATUS OK!
Ratio: 0.874

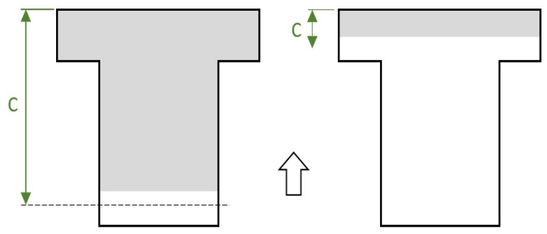
Crack width check (Positive bending moment case)

10.6.1 CRACK CONTROL OF BEAMS



Section input data:
Modulus of elasticity of concrete $E_c = 4500 \cdot \sqrt{f_c} = 4500 \cdot \sqrt{25} = 22500.00 \text{ MPa}$
Modulus of elasticity of steel $E_s = 200000.00 \text{ MPa}$
Modulus Ratio $n = E_s / E_c = 200000 / 22500.00 = 8.89$
Effective tension area of concrete around the main reinforcing $A = 10000.00 \text{ mm}^2$
Cover of the outermost bar $d_c = 50 \text{ mm}$
Given bending moment $M_a = 0.00 \text{ kN-m}$

1. Calculation of neutral axis depth c of cracked section



Calculation is based on iterative process:
- Assume c
- Calculate left part of force equilibrium $A_{comp.} \cdot 0.5 \cdot c + \sum n \cdot A_s \cdot \acute{d}_i + \sum n \cdot A_s \cdot d_i$
- Calculate right part of force equilibrium $A_{comp.} + n \cdot A_s + n \cdot \acute{A}_s$

Searching of neutral axis c (from 650 to 0 mm)

Iter.	c (mm)	As (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	650.00	0.00	72578.12	183953.12	2.535
2	637.00	900.54	70068.47	177789.76	2.537
3	624.00	900.54	67609.52	171727.80	2.540
4	611.00	900.54	65201.27	165767.23	2.542

5	598.00	900.54	62843.72	159908.07	2.545
6	585.00	900.54	60536.87	154150.31	2.546
7	572.00	900.54	58280.72	148493.95	2.548
8	559.00	900.54	56075.27	142938.98	2.549
9	546.00	900.54	53920.52	137485.42	2.550
10	533.00	900.54	51816.47	132133.26	2.550
11	520.00	900.54	49763.12	126882.50	2.550
12	507.00	900.54	47760.47	121733.13	2.549
13	494.00	900.54	45808.52	116685.17	2.547
14	481.00	900.54	43907.27	111738.61	2.545
15	468.00	900.54	42056.72	106893.45	2.542
16	455.00	900.54	40256.87	102149.68	2.537
17	442.00	900.54	38507.72	97507.32	2.532
18	429.00	900.54	36809.27	92966.36	2.526
19	416.00	900.54	35161.52	88526.80	2.518
20	403.00	900.54	33564.47	84188.63	2.508
21	390.00	900.54	32018.12	79951.87	2.497
22	377.00	900.54	30522.47	75816.51	2.484
23	364.00	900.54	29077.52	71782.55	2.469
24	351.00	900.54	27683.27	67849.98	2.451
25	338.00	900.54	26339.72	64018.82	2.431
26	325.00	900.54	25046.87	60289.06	2.407
27	312.00	900.54	23804.72	56660.70	2.380
28	299.00	900.54	22613.27	53133.74	2.350
29	286.00	900.54	21472.52	49708.17	2.315
30	273.00	900.54	20382.47	46384.01	2.276
31	260.00	900.54	19343.12	43161.25	2.231
32	247.00	900.54	18354.47	40039.89	2.181
33	234.00	900.54	17416.52	37019.92	2.126
34	221.00	900.54	16529.27	34101.36	2.063
35	208.00	900.54	15692.72	31284.20	1.994
36	195.00	900.54	14906.87	28568.44	1.916
37	182.00	900.54	14171.72	25954.07	1.831
38	169.00	900.54	13487.27	23441.11	1.738
39	156.00	900.54	12853.52	21029.55	1.636
40	143.00	900.54	12270.47	18719.39	1.526
41	130.00	900.54	11738.12	16510.62	1.407
42	117.00	900.54	11256.47	14403.26	1.280
43	104.00	900.54	10825.52	12397.30	1.145
44	91.00	900.54	9757.67	9837.54	1.008
left part < right part. Updating of iterations					
1	78.00	900.54	8549.32	7316.77	0.856
2	90.74	900.54	9731.68	9783.48	1.005
3	90.48	900.54	9705.77	9729.57	1.002

4	90.22	900.54	9679.93	9675.81	1.000

Final value of c is 90.22 mm and tensioning rebar area is $900.54\ \text{mm}^2$
Working depth of reinforcement $d = 650.00\ \text{mm}$

2. Calculation of stress in tensioning zone of reinforcement

$$f_s = \frac{M_a}{A_s \cdot (d - c/3)} = \frac{0}{900.54 \cdot (650.00 - 90.22/3)} = 0.00\ \text{MPa}$$

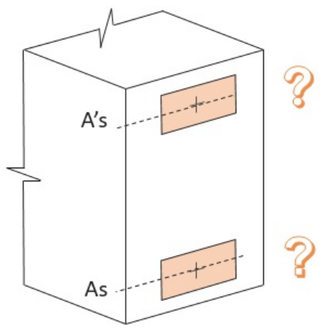
3. Determine the value of z factor (10.6.1)

$$z = f_s \cdot \sqrt[3]{d_c \cdot A} = 0.00 \cdot \sqrt[3]{50.00 \cdot 10000.00} = 0.00\ \text{N/mm}$$

$z = 0.00\ \text{N/mm} \leq z_{lim} = 30000.00\ \text{N/mm}\ (\text{Ratio: } 0.000)$

STATUS OK!
Ratio: 0.000

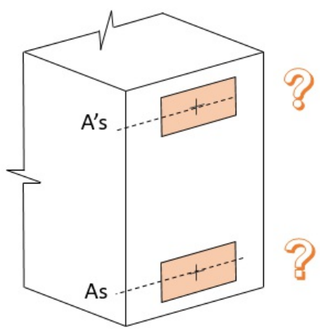
Flexure check (Negative bending moment case)



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 width check (Negative bending moment case)



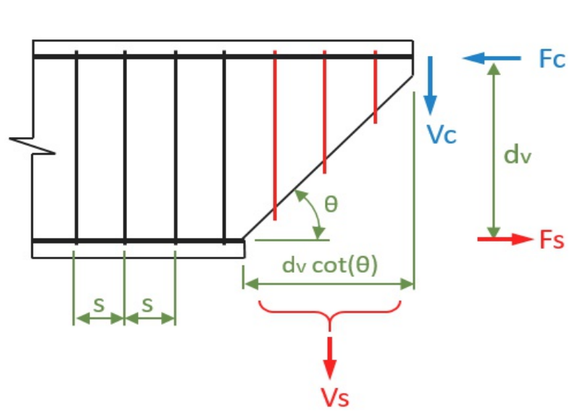
Bottom Reinforcement is absent in the section. Design checks can't be performed. But as acting moment value is equal to zero no need to check.

STATUS OK!

11.2.8.2, 11.3.3, 11.3.4, 11.3.5.1, 11.3.6.3, 11.3.8.1

Shear check

SHEAR FORCE CAPACITY (Members with shear reinforcement)



Section input data:
Mean width of web $b_w = 300\ \text{mm}$
Cross-sectional area of the shear reinforcement $A_v = 157.08\ \text{mm}^2$
Spacing of stirrups $s = 250.00\ \text{mm}$
Given shear force $V = 0.00\ \text{kN}$
Effective shear depth $d_v = max\ \{0.9d, 0.72h\} = 585.00$
Concrete density factor $\lambda = 1$
Concrete resistance factor (8.4.2) $\phi_c = 0.65$

Reinforcement resistance factor (8.4.3) $\phi_s = 0.85$
Shear resistance factor $\beta = 0.18$
Angle of diagonal compressive stresses $\theta = 35$ deg.

1. Calculate Concrete Shear Capacity (11.3.4)

$$V_c = \phi_c \cdot \lambda \cdot \beta \cdot \sqrt{f_c} \cdot b_w \cdot d_v = 0.65 \cdot 1 \cdot 0.180 \cdot \sqrt{25} \cdot 300 \cdot 585.00 = 102.67 \text{ kN}$$

2. Calculate minimum area of shear reinforcement (11.2.8.2)

$$A_{v,min} = 0.06 \cdot \sqrt{f_c} \cdot \frac{b_w \cdot s}{f_y} = 0.06 \cdot \sqrt{25} \cdot \frac{300 \cdot 250}{400} = 56.25 \text{ mm}^2$$

$$A_v = 157.08 \text{ mm}^2 \geq A_{v,min} = 56.25 \text{ mm}^2$$
$$\rightarrow \text{ area of shear reinforcement is satisfied (Ratio: 0.358)}$$

STATUS OK!
Ratio: 0.358

$$V_s = \frac{\phi_s \cdot A_v \cdot f_y \cdot d_v \cdot cot(\theta)}{s} = \frac{0.85 \cdot 157.08 \cdot 400 \cdot 585.00 \cdot cot(35)}{250} = 178.48 \text{ kN}$$

2. Calculate factored shear resistance (11.3.3)

$$V_r = V_c + V_s = 102.67 + 178.48 = 281.15 \text{ kN}$$

Allowed factored shear resistance

$$V_{r,max} = 0.25 \cdot \phi_c \cdot f_c \cdot b_w \cdot d_v = 0.25 \cdot 0.65 \cdot 25 \cdot 300 \cdot 585.00 = 712.97 \text{ kN}$$

$$V_r \leq V_{r,max}$$

$$V = 0.00 \text{ kN} \leq V_r = 281.15 \text{ kN (Ratio: 0.000)}$$

STATUS OK!
Ratio: 0.000