REFERENCES	CALCULATIONS	RESULTS

Code: CSA A23.3-14

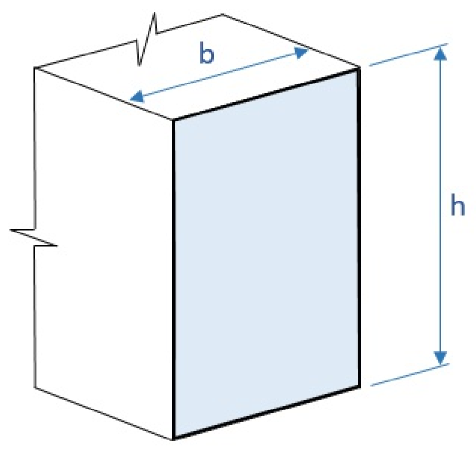
MEMBER #1 (SECTION POSITION 0.0 mm) BEAM DESIGN REPORT

Project details

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

General member design information

Dimentions:



Height $h = 500\ \mathrm{mm}$
Width $b = 300\ \mathrm{mm}$
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 = $0\ \mathrm{kN}$

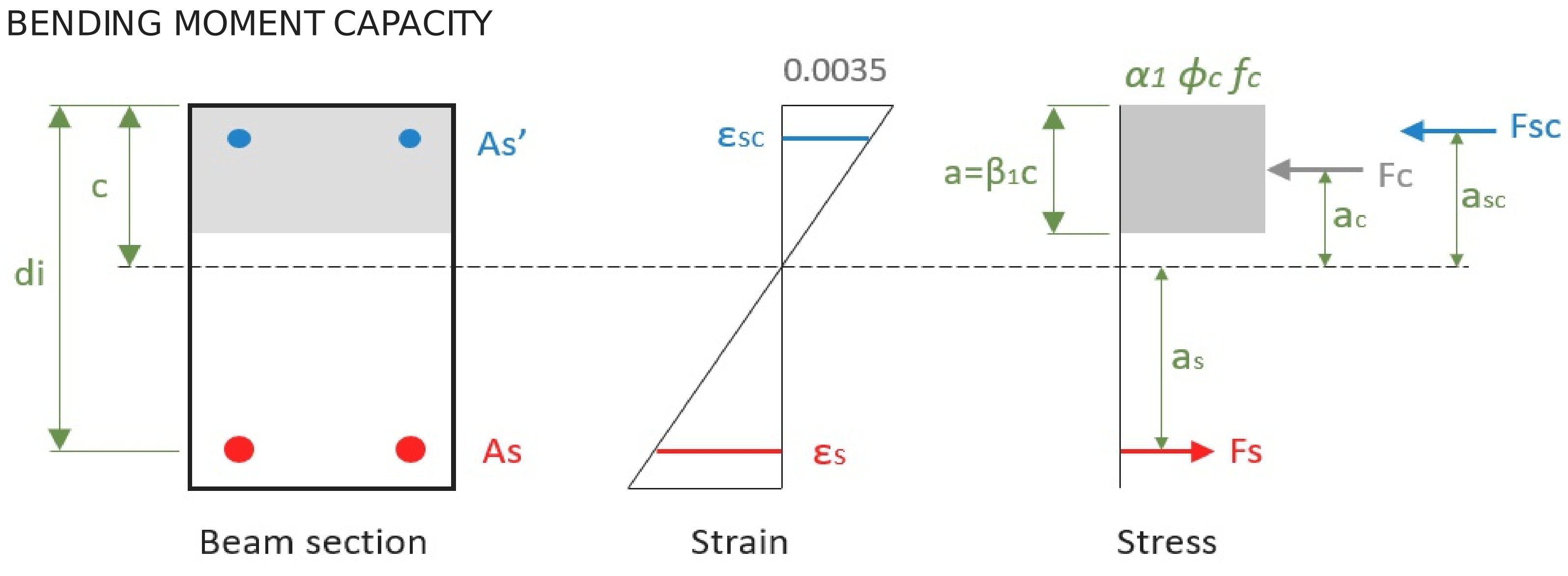
Load Combinations (Serviceability Limit State)

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 $d = 437.5\ \mathrm{mm}$
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

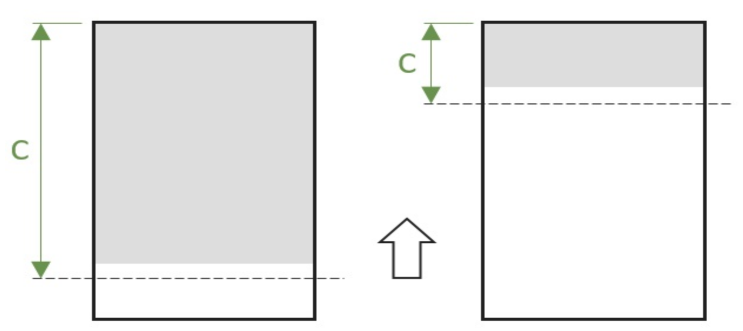
Depth di (mm)	bar diameter (mm)	bar area Asi (mm^2)
437.5	25.23	499.95
437.5	25.23	499.95
437.5	25.23	499.95
437.5	25.23	499.95

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 437.5 to 0 mm)

Iter.	c (mm)	a (mm)	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	437.5	397.0	1572.62	0.00	1572.62	0.00	Infinity
2	428.8	389.1	1541.16	0.00	1541.16	24.28	63.466
3	420.0	381.1	1509.71	0.00	1509.71	49.58	30.451
4	411.3	373.2	1478.26	0.00	1478.26	75.95	19.464
5	402.5	365.3	1446.81	0.00	1446.81	103.47	13.983
6	393.8	357.3	1415.35	0.00	1415.35	132.21	10.705
7	385.0	349.4	1383.90	0.00	1383.90	162.26	8.529
8	376.3	341.4	1352.45	0.00	1352.45	193.70	6.982
9	367.5	333.5	1321.00	0.00	1321.00	226.64	5.829
10	358.8	325.6	1289.55	0.00	1289.55	261.19	4.937
11	350.0	317.6	1258.09	0.00	1258.09	297.47	4.229
12	341.3	309.7	1226.64	0.00	1226.64	335.61	3.655
13	332.5	301.7	1195.19	0.00	1195.19	375.75	3.181
14	323.8	293.8	1163.74	0.00	1163.74	418.07	2.784
15	315.0	285.9	1132.28	0.00	1132.28	462.73	2.447
16	306.3	277.9	1100.83	0.00	1100.83	509.95	2.159
17	297.5	270.0	1069.38	0.00	1069.38	559.94	1.910
18	288.8	262.0	1037.93	0.00	1037.93	612.97	1.693
19	280.0	254.1	1006.47	0.00	1006.47	669.31	1.504
20	271.3	246.2	975.02	0.00	975.02	679.93	1.434
21	262.5	238.2	943.57	0.00	943.57	679.93	1.388

22	253.8	230.3	912.12	0.00	912.12	679.93	1.341
23	245.0	222.3	880.66	0.00	880.66	679.93	1.295
24	236.3	214.4	849.21	0.00	849.21	679.93	1.249
25	227.5	206.5	817.76	0.00	817.76	679.93	1.203
26	218.8	198.5	786.31	0.00	786.31	679.93	1.156
27	210.0	190.6	754.86	0.00	754.86	679.93	1.110
28	201.3	182.6	723.40	0.00	723.40	679.93	1.064
29	192.5	174.7	691.95	0.00	691.95	679.93	1.018
(Fc + Fcs) < Fs. Updating of iterations							
1	183.8	166.8	660.50	0.00	660.50	679.93	0.971
2	192.3	174.5	691.32	0.00	691.32	679.93	1.017
3	192.1	174.4	690.69	0.00	690.69	679.93	1.016
4	192.0	174.2	690.06	0.00	690.06	679.93	1.015
5	191.8	174.1	689.43	0.00	689.43	679.93	1.014
6	191.6	173.9	688.81	0.00	688.81	679.93	1.013
7	191.4	173.7	688.18	0.00	688.18	679.93	1.012
8	191.3	173.6	687.55	0.00	687.55	679.93	1.011
9	191.1	173.4	686.92	0.00	686.92	679.93	1.010
10	190.9	173.3	686.29	0.00	686.29	679.93	1.009
11	190.7	173.1	685.66	0.00	685.66	679.93	1.008
12	190.6	172.9	685.03	0.00	685.03	679.93	1.008
13	190.4	172.8	684.40	0.00	684.40	679.93	1.007
14	190.2	172.6	683.77	0.00	683.77	679.93	1.006
15	190.0	172.5	683.14	0.00	683.14	679.93	1.005
16	189.9	172.3	682.52	0.00	682.52	679.93	1.004
17	189.7	172.2	681.89	0.00	681.89	679.93	1.003
18	189.5	172.0	681.26	0.00	681.26	679.93	1.002
19	189.3	171.8	680.63	0.00	680.63	679.93	1.001
20	189.2	171.7	680.00	0.00	680.00	679.93	1.000
21	189.0	171.5	679.37	0.00	679.37	679.93	0.999

Final value of c is 189.00 mm, flexural tension reinforcement area is 1999.80 mm^2 and flexural compression reinforcement area is 0.00 mm^2
Working depth of reinforcement $d = 437.50\ \mathrm{mm}$

2. Calculation of moment resistance M_r

$$M_r = F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s = 70.14 + 0.00 + 168.96 = 239.10\ \text{kN-m}$$

$$M = 0.00\ \text{kN-m} \leq M_r = 239.10\ \text{kN-m (Ratio: 0.000)}$$

STATUS OK!
Ratio: 0.000

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

Width of tension zone $b_t = 300\ \mathrm{mm}$

$$A_{st,min} = \frac{0.2 \cdot \sqrt{f_c}}{f_y} \cdot b_t \cdot h = \frac{0.2 \cdot \sqrt{25}}{400} \cdot 300 \cdot 500 = 375.00\ \mathrm{mm}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$A_{st,max} = 0.04 \cdot b \cdot d = 0.04 \cdot 300 \cdot 437.50 = 5250.00\ \mathrm{mm}^2$$

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

$$A_{st} = 1999.80 \text{ mm}^2 \leq A_{st,max} = 5250.00 \text{ mm}^2 \text{ (Ratio: 0.381)}$$

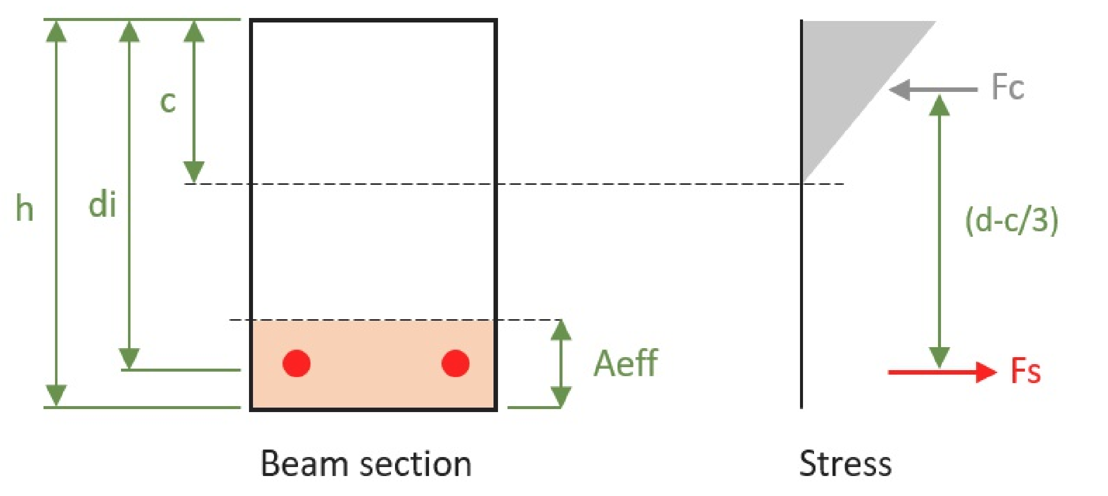
STATUS OK!
Ratio: 0.381

$$A_{st} = 1999.80 \text{ mm}^2 \geq A_{st,min} = 375.00 \text{ mm}^2 \text{ (Ratio: 0.188)}$$

STATUS OK!
Ratio: 0.188

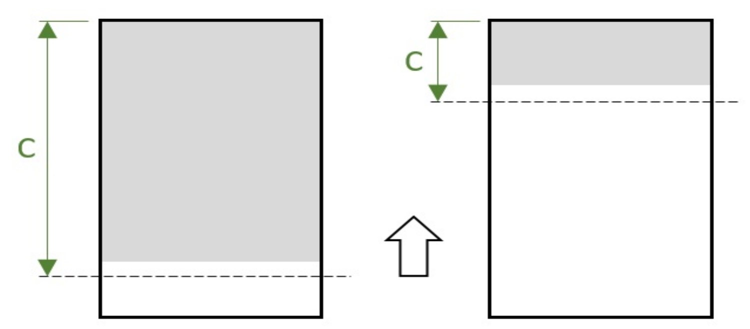
Crack width check (Positive bending moment case)

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 $A = 9375.00 \text{ mm}^2$
Cover of the outermost bar $d_c = 62.5 \text{ mm}$
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 437.5 to 0 mm)

Iter.	c (mm)	As (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	437.50	0.00	36487.94	65198.88	1.787
2	428.75	1999.80	35350.98	62769.43	1.776
3	420.00	1999.80	34237.00	60385.92	1.764
4	411.25	1999.80	33145.98	58048.35	1.751
5	402.50	1999.80	32077.94	55756.71	1.738
6	393.75	1999.80	31032.86	53511.02	1.724
7	385.00	1999.80	30010.75	51311.26	1.710
8	376.25	1999.80	29011.61	49157.44	1.694
9	367.50	1999.80	28035.44	47049.56	1.678
10	358.75	1999.80	27082.23	44987.61	1.661
11	350.00	1999.80	26152.00	42971.60	1.643
12	341.25	1999.80	25244.73	41001.53	1.624
13	332.50	1999.80	24360.44	39077.39	1.604
14	323.75	1999.80	23499.11	37199.20	1.583

15	315.00	1999.80	22660.75	35366.94	1.561
16	306.25	1999.80	21845.36	33580.62	1.537
17	297.50	1999.80	21052.94	31840.24	1.512
18	288.75	1999.80	20283.48	30145.79	1.486
19	280.00	1999.80	19537.00	28497.28	1.459
20	271.25	1999.80	18813.48	26894.71	1.430
21	262.50	1999.80	18112.94	25338.08	1.399
22	253.75	1999.80	17435.36	23827.38	1.367
23	245.00	1999.80	16780.75	22362.62	1.333
24	236.25	1999.80	16149.11	20943.80	1.297
25	227.50	1999.80	15540.44	19570.92	1.259
26	218.75	1999.80	14954.73	18243.97	1.220
27	210.00	1999.80	14392.00	16962.96	1.179
28	201.25	1999.80	13852.23	15727.89	1.135
29	192.50	1999.80	13335.44	14538.75	1.090
30	183.75	1999.80	12841.61	13395.56	1.043
left part < right part. Updating of iterations					
1	175.00	1999.80	12370.75	12298.30	0.994
2	183.57	1999.80	12831.97	13373.16	1.042
3	183.40	1999.80	12822.33	13350.79	1.041
4	183.22	1999.80	12812.71	13328.43	1.040
5	183.05	1999.80	12803.10	13306.09	1.039
6	182.87	1999.80	12793.49	13283.77	1.038
7	182.70	1999.80	12783.89	13261.46	1.037
8	182.52	1999.80	12774.31	13239.18	1.036
9	182.35	1999.80	12764.73	13216.91	1.035
10	182.17	1999.80	12755.16	13194.66	1.034
11	182.00	1999.80	12745.60	13172.43	1.033
12	181.82	1999.80	12736.05	13150.22	1.033
13	181.65	1999.80	12726.51	13128.03	1.032
14	181.47	1999.80	12716.98	13105.85	1.031
15	181.30	1999.80	12707.45	13083.70	1.030
16	181.12	1999.80	12697.94	13061.56	1.029
17	180.95	1999.80	12688.44	13039.44	1.028
18	180.77	1999.80	12678.94	13017.34	1.027
19	180.60	1999.80	12669.45	12995.25	1.026
20	180.42	1999.80	12659.98	12973.19	1.025
21	180.25	1999.80	12650.51	12951.14	1.024
22	180.07	1999.80	12641.05	12929.11	1.023
23	179.90	1999.80	12631.60	12907.11	1.022
24	179.72	1999.80	12622.16	12885.11	1.021
25	179.55	1999.80	12612.73	12863.14	1.020
26	179.37	1999.80	12603.31	12841.19	1.019
27	179.20	1999.80	12593.90	12819.25	1.018

28	179.02	1999.80	12584.49	12797.33	1.017
29	178.85	1999.80	12575.10	12775.43	1.016
30	178.67	1999.80	12565.71	12753.55	1.015
31	178.50	1999.80	12556.34	12731.69	1.014
32	178.32	1999.80	12546.97	12709.85	1.013
33	178.15	1999.80	12537.61	12688.02	1.012
34	177.97	1999.80	12528.27	12666.21	1.011
35	177.80	1999.80	12518.93	12644.42	1.010
36	177.62	1999.80	12509.60	12622.65	1.009
37	177.45	1999.80	12500.28	12600.90	1.008
38	177.27	1999.80	12490.96	12579.17	1.007
39	177.10	1999.80	12481.66	12557.45	1.006
40	176.92	1999.80	12472.37	12535.76	1.005
41	176.75	1999.80	12463.08	12514.08	1.004
42	176.57	1999.80	12453.81	12492.42	1.003
43	176.40	1999.80	12444.54	12470.77	1.002
44	176.22	1999.80	12435.29	12449.15	1.001
45	176.05	1999.80	12426.04	12427.55	1.000
46	175.87	1999.80	12416.80	12405.96	0.999

Final value of c is 175.87 mm and tensioning rebar area is $1999.80\ \mathrm{mm}^2$
Working depth of reinforcement $d = 437.50\ \mathrm{mm}$

2. Calculation of stress in tensioning zone of reinforcement

$$f_s = \frac{M_a}{A_s \cdot (d - c/3)} = \frac{0}{1999.80 \cdot (437.50 - 175.87/3)} = 0.00\ \mathrm{MPa}$$

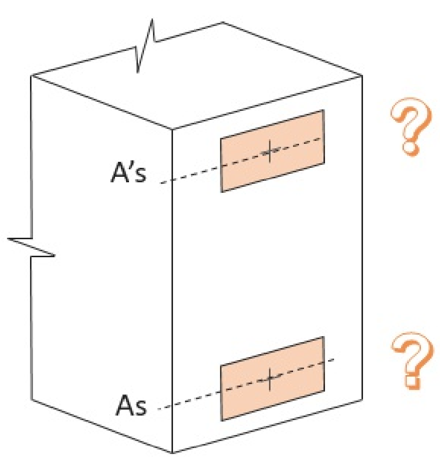
3. Determine the value of z factor (10.6.1)

$$z = f_s \cdot \sqrt[3]{d_c \cdot A} = 0.00 \cdot \sqrt[3]{62.50 \cdot 9375.00} = 0.00\ \mathrm{N/mm}$$

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

STATUS OK!
Ratio: 0.000

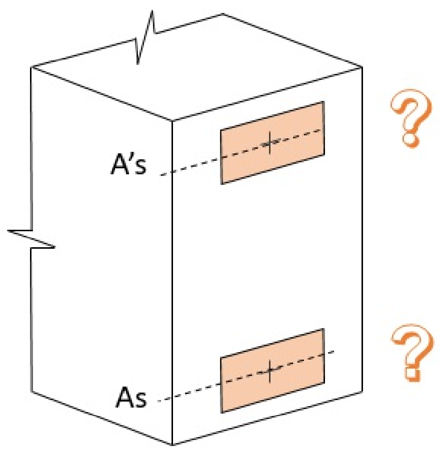
Flexure check (Negative bending moment case)



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)



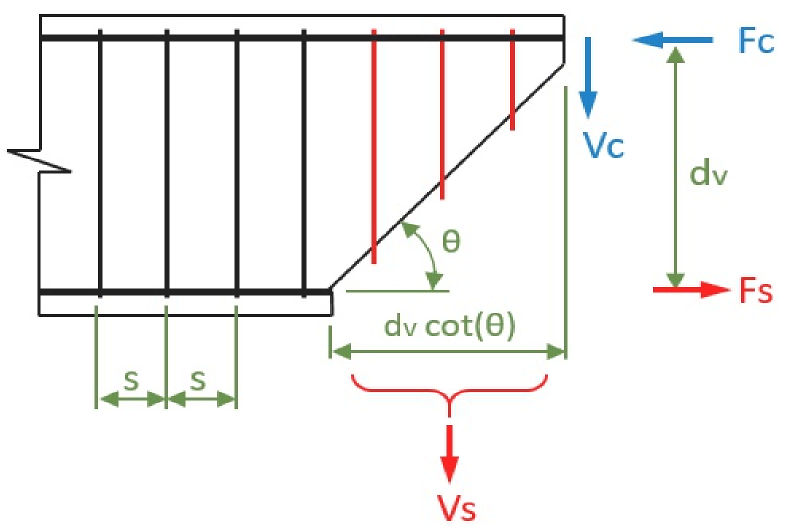
Bottom Reinforcement is absent in the section. Design checks can't be performed. But as acting moment value is equal to zero no need to check.

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 $b_w = 300\ \mathrm{mm}$
Cross-sectional area of the shear reinforcement $A_v = 199.87\ \mathrm{mm}^2$
Spacing of stirrups $s = 250.00\ \mathrm{mm}$
Given shear force $V = 0.00\ \mathrm{kN}$
Effective shear depth $d_v = max\left\{0.9d, 0.72h\right\} = 393.75$
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 $\theta = 35\ \mathrm{deg.}$

1. Calculate Concrete Shear Capacity (11.3.4)

$$V_c = \phi_c \cdot \lambda \cdot \beta \cdot \sqrt{f_c} \cdot b_w \cdot d_v = 0.65 \cdot 1 \cdot 0.180 \cdot \sqrt{25} \cdot 300 \cdot 393.75 = 69.10\ \mathrm{kN}$$

2. Calculate minimum area of shear reinforcement (11.2.8.2)

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

$$A_v = 199.87\ \mathrm{mm}^2 \geq A_{v,min} = 56.25\ \mathrm{mm}^2$$
$$\rightarrow \text{ area of shear reinforcement is satisfied (Ratio: 0.281)}$$

STATUS OK!
Ratio: 0.281

$$V_s = \frac{\phi_s \cdot A_v \cdot f_y \cdot d_v \cdot cot(\theta)}{s} = \frac{0.85 \cdot 199.87 \cdot 400 \cdot 393.75 \cdot cot(35)}{250} = 152.85\ \mathrm{kN}$$

2. Calculate factored shear resistance (11.3.3)

$$V_r = V_c + V_s = 69.10 + 152.85 = 221.95\ \mathrm{kN}$$

Allowed factored shear resistance

$$V_{r,max} = 0.25 \cdot \phi_c \cdot f_c \cdot b_w \cdot d_v = 0.25 \cdot 0.65 \cdot 25 \cdot 300 \cdot 393.75 = 479.88\ \mathrm{kN}$$

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

$$V = 0.00\ \mathrm{kN} \leq V_r = 221.95\ \mathrm{kN}\ \text{(Ratio: 0.000)}$$

STATUS OK!
Ratio: 0.000