REFERENCES	CALCULATIONS	RESULTS

Code: CSA A23.3-14

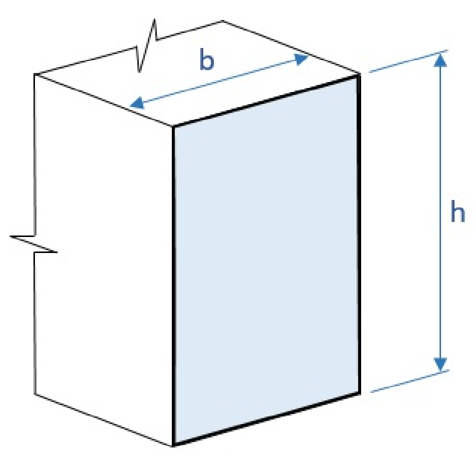
MEMBER #1 (SECTION POSITION 0.0 mm) BEAM DESIGN REPORT

Project details

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

General member design information

Dimentions:



Height $h = 500 \text{ mm}$
Width $b = 400 \text{ mm}$
Member length = 5000 mm

Material properties:
Concrete strength $f_c = 25 \text{ MPa}$
Steel strength of longitudinal rebar $f_y = 400 \text{ MPa}$
Steel strength of shear rebar $f_{yt} = 400 \text{ MPa}$
Limit crack control parameter $z_{lim} = 30000 \text{ N/mm}$

Load Combinations (Ultimate Limit State)

For axial force in section:
LC1: USER = 0 kN

For bending moment in section:
LC1: USER = 0 kN-m

For shear force in section:
LC1: USER = 0 kN

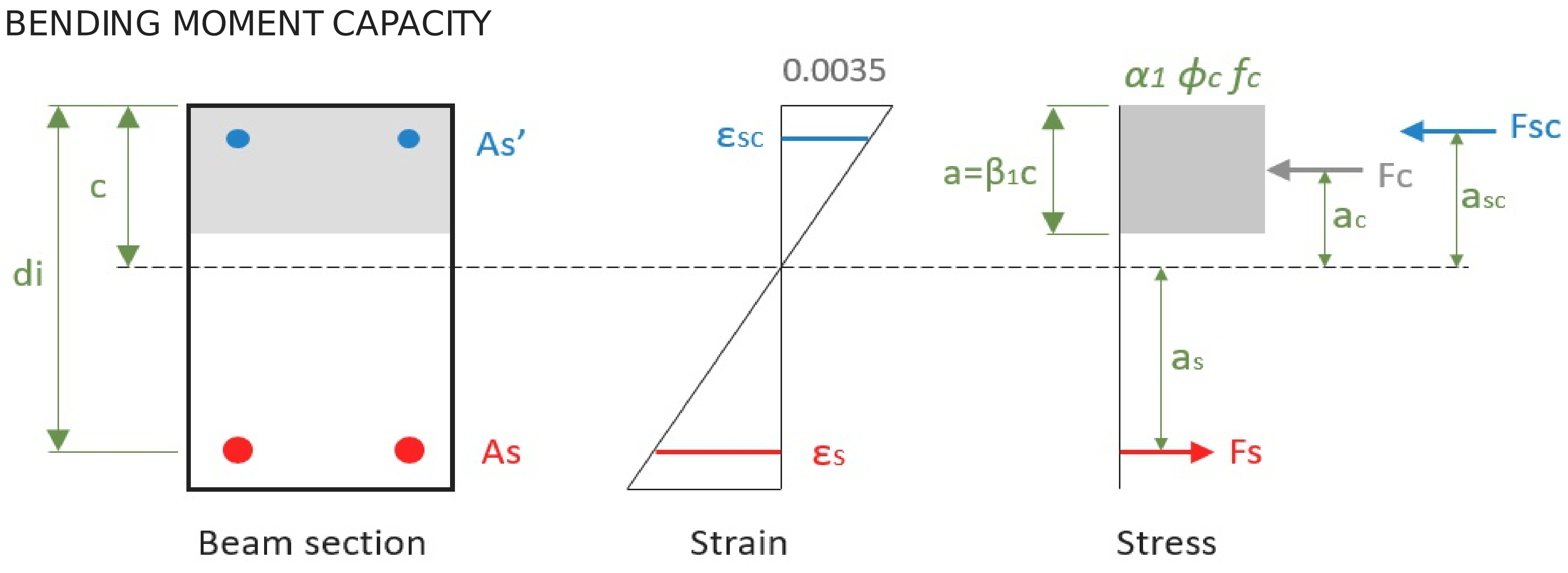
Load Combinations (Serviceability Limit State)

For bending moment in section:
LC1: USER = 0 kN-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 $d = 450 \text{ mm}$
Given bending moment $M = 0.00 \text{ kN-m}$
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

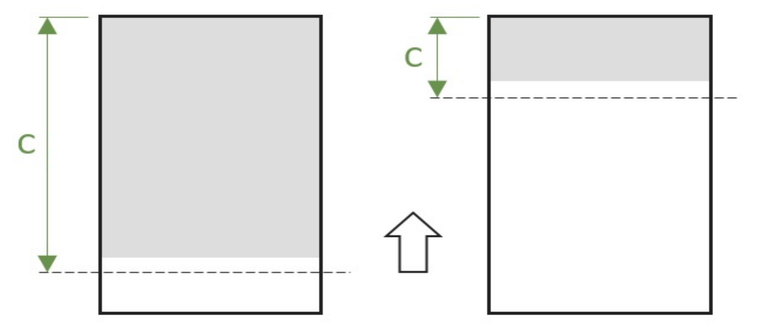
Depth di (mm)	bar diameter (mm)	bar area Asi (mm^2)
450	25.23	499.95
450	25.23	499.95

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 450 to 0 mm)

Iter.	c (mm)	a (mm)	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	450.0	408.4	2156.73	0.00	2156.73	0.00	Infinity
2	441.0	400.2	2113.60	0.00	2113.60	12.14	174.078
3	432.0	392.0	2070.46	0.00	2070.46	24.79	83.523
4	423.0	383.9	2027.33	0.00	2027.33	37.97	53.386
5	414.0	375.7	1984.19	0.00	1984.19	51.73	38.354
6	405.0	367.5	1941.06	0.00	1941.06	66.10	29.363
7	396.0	359.4	1897.92	0.00	1897.92	81.13	23.394
8	387.0	351.2	1854.79	0.00	1854.79	96.85	19.151
9	378.0	343.0	1811.65	0.00	1811.65	113.32	15.987
10	369.0	334.9	1768.52	0.00	1768.52	130.60	13.542
11	360.0	326.7	1725.38	0.00	1725.38	148.74	11.600
12	351.0	318.5	1682.25	0.00	1682.25	167.80	10.025
13	342.0	310.4	1639.12	0.00	1639.12	187.88	8.724
14	333.0	302.2	1595.98	0.00	1595.98	209.03	7.635
15	324.0	294.0	1552.85	0.00	1552.85	231.37	6.712
16	315.0	285.9	1509.71	0.00	1509.71	254.97	5.921
17	306.0	277.7	1466.58	0.00	1466.58	279.97	5.238
18	297.0	269.5	1423.44	0.00	1423.44	306.48	4.644
19	288.0	261.4	1380.31	0.00	1380.31	334.65	4.125
20	279.0	253.2	1337.17	0.00	1337.17	339.97	3.933
21	270.0	245.0	1294.04	0.00	1294.04	339.97	3.806
22	261.0	236.9	1250.90	0.00	1250.90	339.97	3.679
23	252.0	228.7	1207.77	0.00	1207.77	339.97	3.553

24	243.0	220.5	1164.63	0.00	1164.63	339.97	3.426
25	234.0	212.4	1121.50	0.00	1121.50	339.97	3.299
26	225.0	204.2	1078.37	0.00	1078.37	339.97	3.172
27	216.0	196.0	1035.23	0.00	1035.23	339.97	3.045
28	207.0	187.9	992.10	0.00	992.10	339.97	2.918
29	198.0	179.7	948.96	0.00	948.96	339.97	2.791
30	189.0	171.5	905.83	0.00	905.83	339.97	2.664
31	180.0	163.3	862.69	0.00	862.69	339.97	2.538
32	171.0	155.2	819.56	0.00	819.56	339.97	2.411
33	162.0	147.0	776.42	0.00	776.42	339.97	2.284
34	153.0	138.8	733.29	0.00	733.29	339.97	2.157
35	144.0	130.7	690.15	0.00	690.15	339.97	2.030
36	135.0	122.5	647.02	0.00	647.02	339.97	1.903
37	126.0	114.3	603.88	0.00	603.88	339.97	1.776
38	117.0	106.2	560.75	0.00	560.75	339.97	1.649
39	108.0	98.0	517.62	0.00	517.62	339.97	1.523
40	99.0	89.8	474.48	0.00	474.48	339.97	1.396
41	90.0	81.7	431.35	0.00	431.35	339.97	1.269
42	81.0	73.5	388.21	0.00	388.21	339.97	1.142
43	72.0	65.3	345.08	0.00	345.08	339.97	1.015
(Fc + Fcs) < Fs. Updating of iterations							
1	63.0	57.2	301.94	0.00	301.94	339.97	0.888
2	71.8	65.2	344.21	0.00	344.21	339.97	1.012
3	71.6	65.0	343.35	0.00	343.35	339.97	1.010
4	71.5	64.8	342.49	0.00	342.49	339.97	1.007
5	71.3	64.7	341.63	0.00	341.63	339.97	1.005
6	71.1	64.5	340.76	0.00	340.76	339.97	1.002
7	70.9	64.4	339.90	0.00	339.90	339.97	1.000

Final value of c is 70.92 mm, flexural tension reinforcement area is 999.90 mm^2 and flexural compression reinforcement area is 0.00 mm^2
Working depth of reinforcement $d = 450.00\ \mathrm{mm}$

2. Calculation of moment resistance M_r

$$M_r = F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s = 13.17 + 0.00 + 128.87 = 142.04\ \text{kN-m}$$

$$M = 0.00\ \text{kN-m} \leq M_r = 142.04\ \text{kN-m (Ratio: 0.000)}$$

STATUS OK!
Ratio: 0.000

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

Width of tension zone $b_t = 400\ \mathrm{mm}$

$$A_{st,min} = \frac{0.2 \cdot \sqrt{f_c}}{f_y} \cdot b_t \cdot h = \frac{0.2 \cdot \sqrt{25}}{400} \cdot 400 \cdot 500 = 500.00\ \mathrm{mm}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$A_{st,max} = 0.04 \cdot b \cdot d = 0.04 \cdot 400 \cdot 450.00 = 7200.00\ \mathrm{mm}^2$$

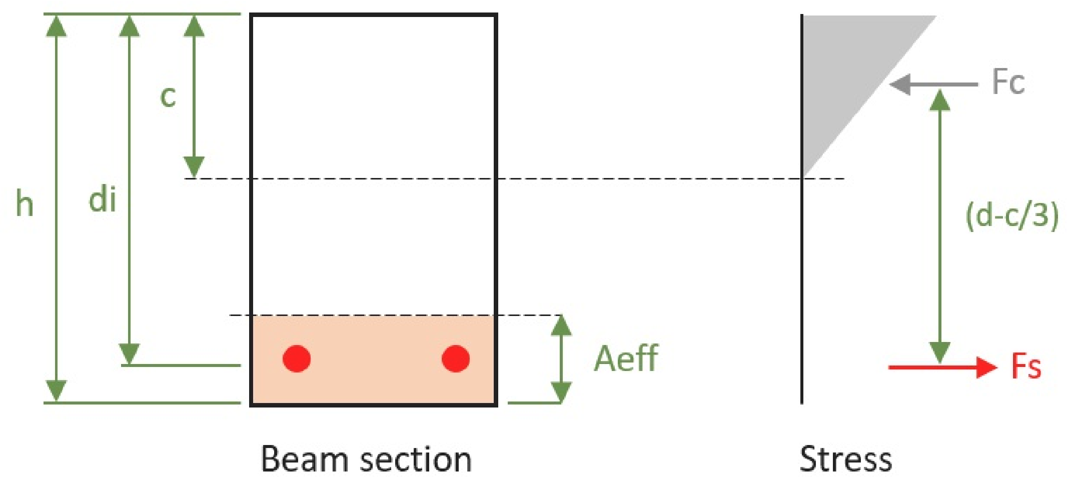
5. Check of required flexural tension reinforcement in a beam section

$A_{st} = 999.90 \text{ mm}^2 \leq A_{st,max} = 7200.00 \text{ mm}^2$ (Ratio: 0.139) **STATUS OK! Ratio: 0.139**

$A_{st} = 999.90 \text{ mm}^2 \geq A_{st,min} = 500.00 \text{ mm}^2$ (Ratio: 0.500) **STATUS OK! Ratio: 0.500**

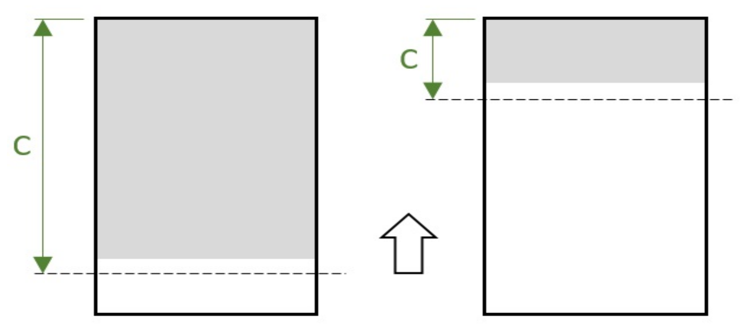
Crack width check (Positive bending moment case)

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 $A = 20000.00 \text{ mm}^2$
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 450 to 0 mm)

Iter.	c (mm)	As (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	450.00	0.00	44499.60	84999.60	1.910
2	441.00	999.90	42895.80	81712.01	1.905
3	432.00	999.90	41324.40	78489.22	1.899
4	423.00	999.90	39785.40	75331.22	1.893
5	414.00	999.90	38278.80	72238.03	1.887
6	405.00	999.90	36804.60	69209.64	1.880
7	396.00	999.90	35362.80	66246.05	1.873
8	387.00	999.90	33953.40	63347.26	1.866
9	378.00	999.90	32576.40	60513.26	1.858
10	369.00	999.90	31231.80	57744.07	1.849
11	360.00	999.90	29919.60	55039.68	1.840
12	351.00	999.90	28639.80	52400.09	1.830
13	342.00	999.90	27392.40	49825.30	1.819
14	333.00	999.90	26177.40	47315.30	1.807
15	324.00	999.90	24994.80	44870.11	1.795

16	315.00	999.90	23844.60	42489.72	1.782
17	306.00	999.90	22726.80	40174.13	1.768
18	297.00	999.90	21641.40	37923.34	1.752
19	288.00	999.90	20588.40	35737.34	1.736
20	279.00	999.90	19567.80	33616.15	1.718
21	270.00	999.90	18579.60	31559.76	1.699
22	261.00	999.90	17623.80	29568.17	1.678
23	252.00	999.90	16700.40	27641.38	1.655
24	243.00	999.90	15809.40	25779.38	1.631
25	234.00	999.90	14950.80	23982.19	1.604
26	225.00	999.90	14124.60	22249.80	1.575
27	216.00	999.90	13330.80	20582.21	1.544
28	207.00	999.90	12569.40	18979.42	1.510
29	198.00	999.90	11840.40	17441.42	1.473
30	189.00	999.90	11143.80	15968.23	1.433
31	180.00	999.90	10479.60	14559.84	1.389
32	171.00	999.90	9847.80	13216.25	1.342
33	162.00	999.90	9248.40	11937.46	1.291
34	153.00	999.90	8681.40	10723.46	1.235
35	144.00	999.90	8146.80	9574.27	1.175
36	135.00	999.90	7644.60	8489.88	1.111
37	126.00	999.90	7174.80	7470.29	1.041
left part < right part. Updating of iterations					
1	117.00	999.90	6737.40	6515.50	0.967
2	125.82	999.90	7165.73	7450.56	1.040
3	125.64	999.90	7156.68	7430.85	1.038
4	125.46	999.90	7147.64	7411.17	1.037
5	125.28	999.90	7138.62	7391.52	1.035
6	125.10	999.90	7129.60	7371.89	1.034
7	124.92	999.90	7120.60	7352.29	1.033
8	124.74	999.90	7111.61	7332.72	1.031
9	124.56	999.90	7102.64	7313.17	1.030
10	124.38	999.90	7093.68	7293.64	1.028
11	124.20	999.90	7084.73	7274.15	1.027
12	124.02	999.90	7075.79	7254.67	1.025
13	123.84	999.90	7066.87	7235.23	1.024
14	123.66	999.90	7057.96	7215.81	1.022
15	123.48	999.90	7049.06	7196.41	1.021
16	123.30	999.90	7040.18	7177.05	1.019
17	123.12	999.90	7031.31	7157.70	1.018
18	122.94	999.90	7022.45	7138.39	1.017
19	122.76	999.90	7013.60	7119.10	1.015
20	122.58	999.90	7004.77	7099.83	1.014
21	122.40	999.90	6995.95	7080.60	1.012

22	122.22	999.90	6987.15	7061.38	1.011
23	122.04	999.90	6978.35	7042.20	1.009
24	121.86	999.90	6969.57	7023.04	1.008
25	121.68	999.90	6960.80	7003.90	1.006
26	121.50	999.90	6952.05	6984.79	1.005
27	121.32	999.90	6943.31	6965.71	1.003
28	121.14	999.90	6934.58	6946.65	1.002
29	120.96	999.90	6925.86	6927.62	1.000
30	120.78	999.90	6917.16	6908.62	0.999

Final value of c is 120.78 mm and tensioning rebar area is $999.90\ \mathrm{mm}^2$
Working depth of reinforcement $d = 450.00\ \mathrm{mm}$

2. Calculation of stress in tensioning zone of reinforcement

$$f_s = \frac{M_a}{A_s \cdot (d - c/3)} = \frac{0}{999.90 \cdot (450.00 - 120.78/3)} = 0.00\ \mathrm{MPa}$$

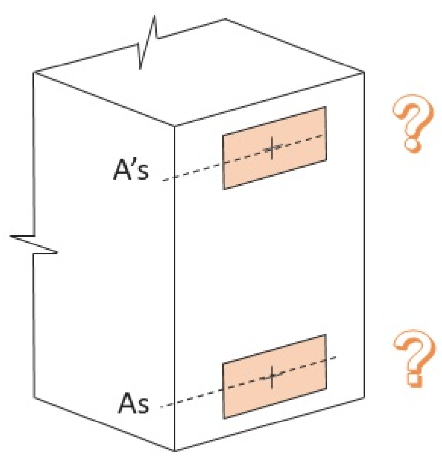
3. Determine the value of z factor (10.6.1)

$$z = f_s \cdot \sqrt[3]{d_c \cdot A} = 0.00 \cdot \sqrt[3]{50.00 \cdot 20000.00} = 0.00\ \mathrm{N/mm}$$

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

STATUS OK!
Ratio: 0.000

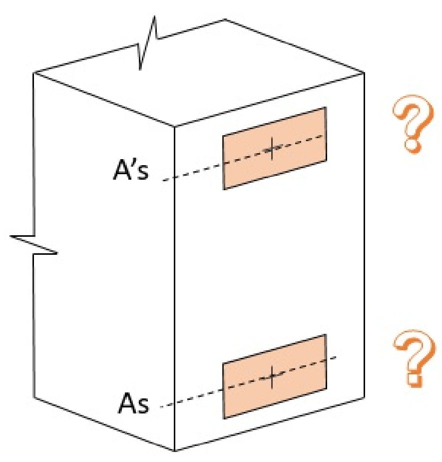
Flexure check (Negative bending moment case)



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)



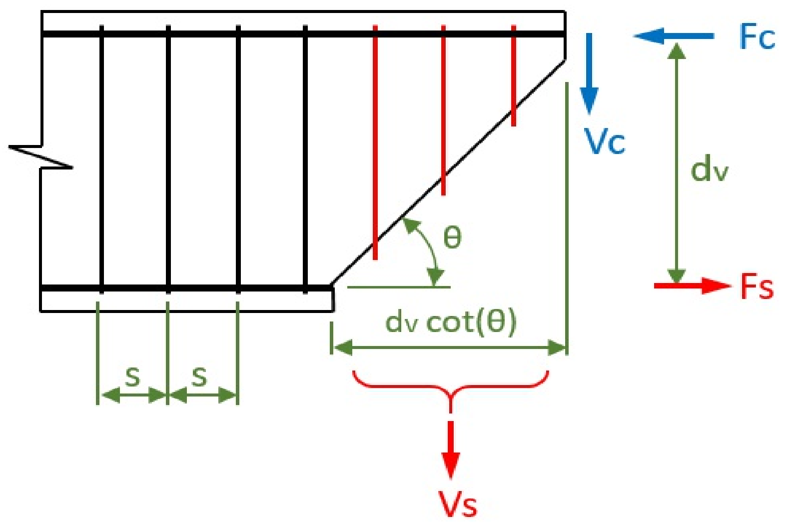
Bottom Reinforcement is absent in the section. Design checks can't be performed. But as acting moment value is equal to zero no need to check.

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 $b_w = 400\ \mathrm{mm}$
Cross-sectional area of the shear reinforcement $A_v = 157.08\ \mathrm{mm}^2$
Spacing of stirrups $s = 250.00\ \mathrm{mm}$
Given shear force $V = 0.00\ \mathrm{kN}$
Effective shear depth $d_v = max\,\{0.9d, 0.72h\} = 405.00$
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)

$$V_c = \phi_c \cdot \lambda \cdot \beta \cdot \sqrt{f_c} \cdot b_w \cdot d_v = 0.65 \cdot 1 \cdot 0.180 \cdot \sqrt{25} \cdot 400 \cdot 405.00 = 94.77\ \mathrm{kN}$$

2. Calculate minimum area of shear reinforcement (11.2.8.2)

$$A_{v,min} = 0.06 \cdot \sqrt{f_c} \cdot \frac{b_w \cdot s}{f_y} = 0.06 \cdot \sqrt{25} \cdot \frac{400 \cdot 250}{400} = 75.00\ \mathrm{mm}^2$$

$$A_v = 157.08\ \mathrm{mm}^2 \geq A_{v,min} = 75.00\ \mathrm{mm}^2$$
$$\rightarrow \text{ area of shear reinforcement is satisfied (Ratio: 0.477)}$$

STATUS OK!
Ratio: 0.477

$$V_s = \frac{\phi_s \cdot A_v \cdot f_y \cdot d_v \cdot cot(\theta)}{s} = \frac{0.85 \cdot 157.08 \cdot 400 \cdot 405.00 \cdot cot(35)}{250} = 123.56\ \mathrm{kN}$$

2. Calculate factored shear resistance (11.3.3)

$$V_r = V_c + V_s = 94.77 + 123.56 = 218.33\ \mathrm{kN}$$

Allowed factored shear resistance

$$V_{r,max} = 0.25 \cdot \phi_c \cdot f_c \cdot b_w \cdot d_v = 0.25 \cdot 0.65 \cdot 25 \cdot 400 \cdot 405.00 = 658.13\ \mathrm{kN}$$

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

$$V = 0.00\ \mathrm{kN} \leq V_r = 218.33\ \mathrm{kN}\ \text{(Ratio: 0.000)}$$

STATUS OK!
Ratio: 0.000