REFERENCES	CALCULATIONS	RESULTS

Code: ENV 1992-1-1 :1991

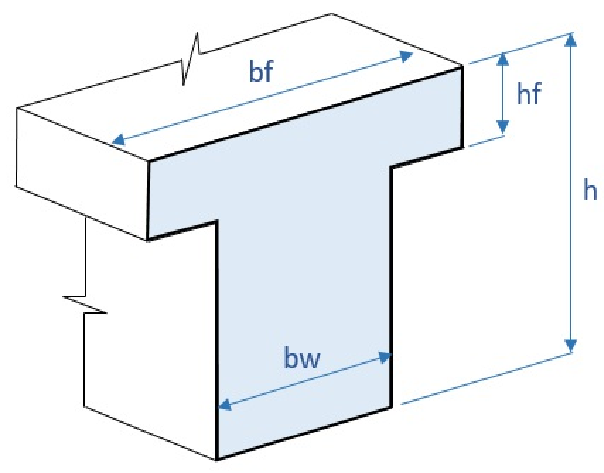
MEMBER #1 (SECTION POSITION 0.0 mm) BEAM DESIGN REPORT

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 $b_f = 450\ \mathrm{mm}$
Flange thickness $h_f = 150\ \mathrm{mm}$
Web width $b_w = 300\ \mathrm{mm}$
Member length = $5000\ \mathrm{mm}$

Material properties:
Concrete strength $f_{ck} = 25\ \mathrm{MPa}$
Steel strength of longitudinal rebar $f_{yk} = 500\ \mathrm{MPa}$
Steel strength of shear rebar $f_{ywk} = 500\ \mathrm{MPa}$
Limiting crack width $\omega_{max} = 0.3\ \mathrm{mm}$

Design Factors and Settings:
Partial safety factor for concrete $\gamma_c = 1.50$
Partial safety factor for rebar $\gamma_s = 1.15$
Long term and unfavorable effects for concrete $\alpha_{cc} = 0.85$

Load Combinations (Ultimate Limit State)

For axial force in section:
LC1: USER = $0\ \mathrm{kN}$

For bending moment in section:
LC1: USER = $0\ \mathrm{kN\text{-}m}$

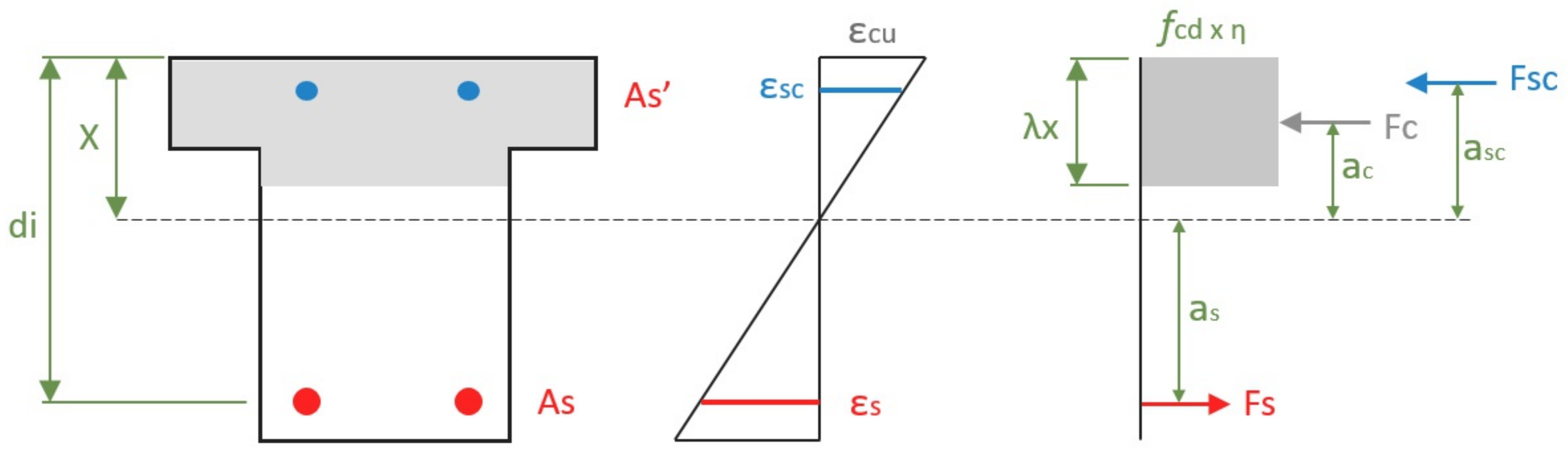
For shear force in section:
LC1: USER = $0\ \mathrm{kN}$

Load Combinations (Serviceability Limit State)

For bending moment in section:
LC1: USER = $0\ \mathrm{kN\text{-}m}$

Flexure check (Positive bending moment case)

BENDING MOMENT CAPACITY



Beam section Strain Stress

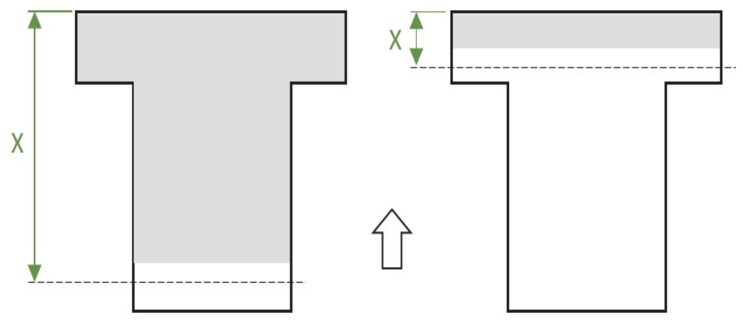
Section input data:
Section concrete area $A_c = 202500\ \mathrm{mm}^2$
Design compressive strength of concrete $f_{cd} = \alpha_{cc} \cdot f_{ck}/\gamma_c = 0.85 \cdot 25/1.5 = 14.17\ \mathrm{MPa}$
Design strength of rebar $f_{yd} = f_{yk}/\gamma_s = 500/1.15 = 434.78\ \mathrm{MPa}$
Design yield strain of rebar $e_y = f_{yd}/E_s = 434.78/200000 = 0.00217$

Ultimate strain in concrete (Table 3) $e_{cu} = 0.00350$
Effective height of the compression zone factor (3.1.7(3)) $\lambda = 0.80$
Effective strength of concrete factor (3.1.7(3)) $\eta = 1.00$
Given bending moment $M_{Ed} = 0.00$ kN-m

Section Rebar

Depth di (mm)	bar diameter (mm)	bar area Asi (mm2)
550	28.73	648.28
550	28.73	648.28
550	28.73	648.28
550	28.73	648.28

1. Calculation of neutral axis depth x



Calculation is based on iterative process:
- Assume x
- Calculate concrete force $F_c = \eta \cdot f_{cd} \cdot \int_{dA} \cdot \lambda \cdot x$
- 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 550 to 0 mm)

Iter.	x (mm)	As (mm2)	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	550.0	0.0	2188.75	0.00	2188.75	0.00	Infinity
2	539.0	2593.1	2151.35	0.00	2151.35	37.04	58.075
3	528.0	2593.1	2113.95	0.00	2113.95	75.63	27.950
4	517.0	2593.1	2076.55	0.00	2076.55	115.86	17.922
5	506.0	2593.1	2039.15	0.00	2039.15	157.84	12.919
6	495.0	2593.1	2001.75	0.00	2001.75	201.69	9.925
7	484.0	2593.1	1964.35	0.00	1964.35	247.53	7.936
8	473.0	2593.1	1926.95	0.00	1926.95	295.50	6.521
9	462.0	2593.1	1889.55	0.00	1889.55	345.75	5.465
10	451.0	2593.1	1852.15	0.00	1852.15	398.46	4.648
11	440.0	2593.1	1814.75	0.00	1814.75	453.80	3.999
12	429.0	2593.1	1777.35	0.00	1777.35	511.97	3.472
13	418.0	2593.1	1739.95	0.00	1739.95	573.22	3.035
14	407.0	2593.1	1702.55	0.00	1702.55	637.77	2.670
15	396.0	2593.1	1665.15	0.00	1665.15	705.90	2.359
16	385.0	2593.1	1627.75	0.00	1627.75	777.94	2.092
17	374.0	2593.1	1590.35	0.00	1590.35	854.20	1.862
18	363.0	2593.1	1552.95	0.00	1552.95	935.09	1.661
19	352.0	2593.1	1515.55	0.00	1515.55	1021.04	1.484
20	341.0	2593.1	1478.15	0.00	1478.15	1112.53	1.329
21	330.0	2593.1	1440.75	0.00	1440.75	1127.44	1.278
22	319.0	2593.1	1403.35	0.00	1403.35	1127.44	1.245

23	308.0	2593.1	1365.95	0.00	1365.95	1127.44	1.212
24	297.0	2593.1	1328.55	0.00	1328.55	1127.44	1.178
25	286.0	2593.1	1291.15	0.00	1291.15	1127.44	1.145
26	275.0	2593.1	1253.75	0.00	1253.75	1127.44	1.112
27	264.0	2593.1	1216.35	0.00	1216.35	1127.44	1.079
28	253.0	2593.1	1178.95	0.00	1178.95	1127.44	1.046
29	242.0	2593.1	1141.55	0.00	1141.55	1127.44	1.013
(Fc + Fcs) < Fs. Updating of iterations							
1	231.0	2593.1	1104.15	0.00	1104.15	1127.44	0.979
2	241.8	2593.1	1140.80	0.00	1140.80	1127.44	1.012
3	241.6	2593.1	1140.05	0.00	1140.05	1127.44	1.011
4	241.3	2593.1	1139.31	0.00	1139.31	1127.44	1.011
5	241.1	2593.1	1138.56	0.00	1138.56	1127.44	1.010
6	240.9	2593.1	1137.81	0.00	1137.81	1127.44	1.009
7	240.7	2593.1	1137.06	0.00	1137.06	1127.44	1.009
8	240.5	2593.1	1136.31	0.00	1136.31	1127.44	1.008
9	240.2	2593.1	1135.57	0.00	1135.57	1127.44	1.007
10	240.0	2593.1	1134.82	0.00	1134.82	1127.44	1.007
11	239.8	2593.1	1134.07	0.00	1134.07	1127.44	1.006
12	239.6	2593.1	1133.32	0.00	1133.32	1127.44	1.005
13	239.4	2593.1	1132.57	0.00	1132.57	1127.44	1.005
14	239.1	2593.1	1131.83	0.00	1131.83	1127.44	1.004
15	238.9	2593.1	1131.08	0.00	1131.08	1127.44	1.003
16	238.7	2593.1	1130.33	0.00	1130.33	1127.44	1.003
17	238.5	2593.1	1129.58	0.00	1129.58	1127.44	1.002
18	238.3	2593.1	1128.83	0.00	1128.83	1127.44	1.001
19	238.0	2593.1	1128.09	0.00	1128.09	1127.44	1.001
20	237.8	2593.1	1127.34	0.00	1127.34	1127.44	1.000

Final value of x is 237.82 mm and tensioning rebar area is 2593.12 mm2
Working depth of reinforcement $d = 550.00 \text{ mm}$

2. Calculation moment resistance M_{Rd}

$$M_{Rd} = F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s = 167.28 + 0.00 + 351.97 = 519.24 \text{ kN-m}$$

$$M_{Ed} = 0.00 \text{ kN-m} \leq M_{Rd} = 519.24 \text{ kN-m}$$

STATUS OK!

9.2.1.1 (3)
9.2.1.1(1)

3. Calculation of maximum allowed longitudinal reinforcement (9.2.1.1 (3))

$$f_{ck} = 25 \text{ MPa} \leq 50 \text{ MPa}$$

$$f_{ctm} = 0.3 \cdot f_{ck}^{2/3} = 0.3 \cdot 25^{2/3} = 2.56 \text{ MPa}$$

$$A_{s,max} = 0.04 \cdot A_c = 0.04 \cdot 202500 = 8100 \text{ mm}^2$$

4. Calculation of minimum allowed longitudinal reinforcement (9.2.1.1(1))

$$A_{s,min1} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d = 0.26 \cdot \frac{2.56}{500} \cdot 300 \cdot 550.00 = 220.07 \text{ mm}^2$$

$$A_{s,min2} = 0.0013 \cdot b_t \cdot d = 0.0013 \cdot 300 \cdot 550.00 = 247.500 \text{ mm}^2$$

$$A_{s,min} = max\left[A_{s,min1}, A_{s,min2}\right] = 247.50 \text{ mm}^2$$

Check of allowed longitudinal reinforcement

$$A_s = 2593.12 \text{ mm}^2 \leq A_{s,max} = 8100.00 \text{ mm}^2$$

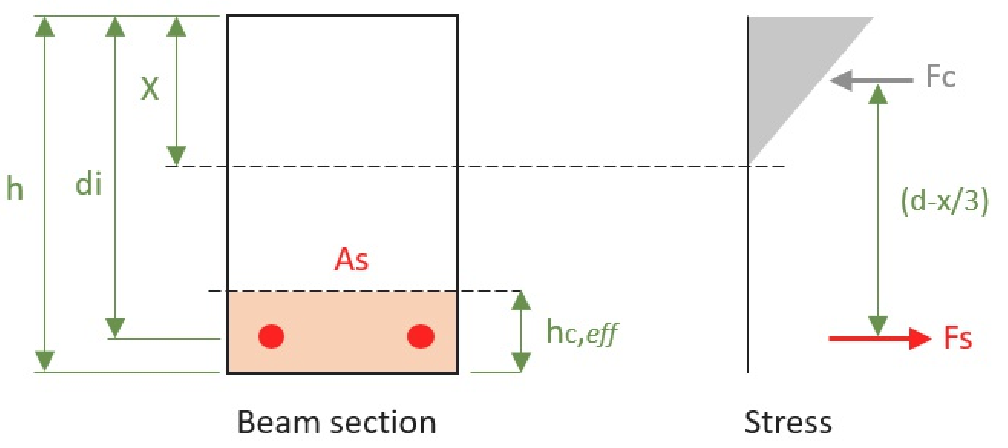
STATUS OK!

$$A_s = 2593.12 \text{ mm}^2 \geq A_{s,min} = 247.50 \text{ mm}^2$$

STATUS OK!

Crack width check (Positive bending moment case)

7.3.4(2) CRACK WIDTH CAPACITY



Section input data:
Depth to the outermost tension side of reinforcement $d_t = 550$ mm
Section concrete area $A_c = 202500 \text{ mm}^2$
Age of concrete at loading $t_0 = 3$ days
Age of concrete at the moment considered $t = 10000$ days
Relative humidity $RH = 70$ %
Given bending moment $M = 0.00$ kN-m

1. Creep coefficient $\phi(t, t_0)$ (ANNEX B)

$$f_{cm} = f_{ck} + 8 = 25 + 8 = 33 \text{ MPa}$$

$$E_{cm} = 22 \cdot \frac{f_{cm}}{10}^{0.3} \cdot 1000 = 22 \cdot \frac{33}{10}^{0.3} \cdot 1000 = 31475.81 \text{ MPa}$$

Notional size of the member

$$h_0 = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 202500.00}{2100} = 192.86 \text{ mm}$$

Factor to allow for the effect of relative humidity on the notional creep coefficient

$$f_{cm} \leq 35\text{MPa} \rightarrow \phi_{RH} = 1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h_0}} = 1 + \frac{1 - \frac{70}{100}}{0.1 \cdot \sqrt[3]{192.86}} = 1.52$$

Factor to allow for the effect of concrete strength on the notional creep coefficient

$$\beta(f_{cm}) = \frac{16.8}{\sqrt{f_{cm}}} = \frac{16.8}{\sqrt{33}} = 2.92$$

Factor to allow for the effect of concrete age at loading on the notional creep coefficient

$$\beta(t_0) = \frac{1}{0.1 + t_0^{0.2}} = \frac{1}{0.1 + 3^{0.2}} = 0.74$$

Notional creep coefficient

$$\phi_0 = \phi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) = 1.52 \cdot 2.92 \cdot 0.74 = 3.30$$

Coefficient depending on the relative humidity and the notional member size

$$\beta_H = 1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h_0 + 250 = 1.5 \cdot \left[1 + (0.012 \cdot 70)^{18}\right] \cdot 192.86 + 250 = 551.83$$

$$\beta_H \leq 1500$$

Coefficient to describe the development of creep with time after loading

$$\beta_c(t,t_0) = \left[\frac{(t-t_0)}{(\beta_H + t - t_0)}\right]^{0.3} = \left[\frac{(10000-3)}{(551.83+10000-3)}\right]^{0.3} = 0.98$$

Creep coefficient

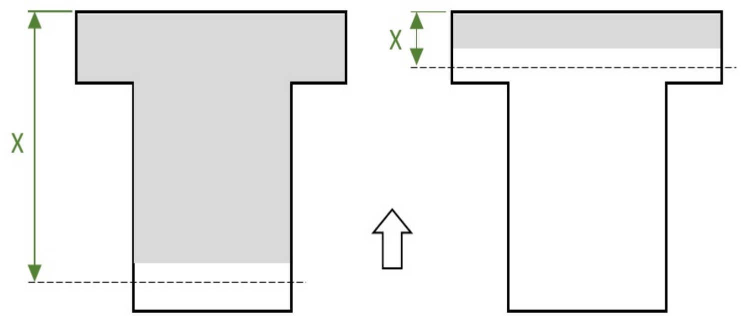
$$\phi(t,t_0) = \phi_0 \cdot \beta_c(t,t_0) = 3.30 \cdot 0.98 = 3.25$$

Effective modulus

$$E_{eff} = \frac{E_{cm}}{1+\phi(t,t_0)} = \frac{31475.81}{1+3.25} = 7408.17 \text{ MPa}$$

$$a_e = \frac{E_s}{E_{eff}} = \frac{200000}{7408.17} = 27.00$$

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)	Asc (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	550.00	0.00	0.00	85566.39	141628.89	1.655
2	539.00	2593.12	0.00	83769.54	137017.61	1.636
3	528.00	2593.12	0.00	82008.99	132478.93	1.615
4	517.00	2593.12	0.00	80284.74	128012.86	1.594
5	506.00	2593.12	0.00	78596.79	123619.38	1.573
6	495.00	2593.12	0.00	76945.14	119298.50	1.550
7	484.00	2593.12	0.00	75329.79	115050.22	1.527
8	473.00	2593.12	0.00	73750.74	110874.54	1.503
9	462.00	2593.12	0.00	72207.99	106771.47	1.479
10	451.00	2593.12	0.00	70701.54	102740.99	1.453
11	440.00	2593.12	0.00	69231.39	98783.11	1.427
12	429.00	2593.12	0.00	67797.54	94897.83	1.400
13	418.00	2593.12	0.00	66399.99	91085.16	1.372
14	407.00	2593.12	0.00	65038.74	87345.08	1.343
15	396.00	2593.12	0.00	63713.79	83677.60	1.313
16	385.00	2593.12	0.00	62425.14	80082.72	1.283
17	374.00	2593.12	0.00	61172.79	76560.44	1.252
18	363.00	2593.12	0.00	59956.74	73110.77	1.219
19	352.00	2593.12	0.00	58776.99	69733.69	1.186
20	341.00	2593.12	0.00	57633.54	66429.21	1.153
21	330.00	2593.12	0.00	56526.39	63197.33	1.118
22	319.00	2593.12	0.00	55455.54	60038.06	1.083
23	308.00	2593.12	0.00	54420.99	56951.38	1.046
24	297.00	2593.12	0.00	53422.74	53937.30	1.010
left part < right part. Updating of iterations						
1	286.00	2593.12	0.00	52460.79	50995.82	0.972
2	296.78	2593.12	0.00	53403.14	53877.76	1.009
3	296.56	2593.12	0.00	53383.56	53818.25	1.008
4	296.34	2593.12	0.00	53364.00	53758.76	1.007
5	296.12	2593.12	0.00	53344.45	53699.31	1.007
6	295.90	2593.12	0.00	53324.91	53639.89	1.006
7	295.68	2593.12	0.00	53305.39	53580.49	1.005
8	295.46	2593.12	0.00	53285.88	53521.12	1.004
9	295.24	2593.12	0.00	53266.39	53461.78	1.004
10	295.02	2593.12	0.00	53246.91	53402.48	1.003
11	294.80	2593.12	0.00	53227.44	53343.20	1.002
12	294.58	2593.12	0.00	53208.00	53283.95	1.001
13	294.36	2593.12	0.00	53188.56	53224.72	1.001
14	294.14	2593.12	0.00	53169.14	53165.53	1.00

Value of x is 294.14 mm
Tensioning rebar area $A_s = 2593.12\ \mathrm{mm}^2$
Compression rebar area $A_{sc} = 0.00\ \mathrm{mm}^2$

Working depth of reinforcement $d = 550.00\ \mathrm{mm}$

3. Calculation of effective strain

$$\sigma_s = \frac{M}{A_s \cdot (d - x/3)} = \frac{0}{2593.12 \cdot (550 - 294.14/3)} = 0.00\ \mathrm{MPa}$$

$$k_t = 0.4 \text{ as long term loading}$$

$$f_{ck} = 25\ \mathrm{MPa} \leq 50\ \mathrm{MPa}$$

$$f_{ctm} = 0.3 \cdot f_{ck}^{2/3} = 0.3 \cdot 25^{2/3} = 2.56\ \mathrm{MPa}$$

$$f_{ct,eff} = f_{ctm}$$

$$a_e = \frac{E_s}{E_{cm}} = \frac{200000}{31475.81} = 6.35$$

$$h_{eff} = max\left[2.5 \cdot (h - d), \frac{h - x}{3}, \frac{h}{2}\right] = max\,[125.00, 101.95, 300.00] = 101.95\ \mathrm{mm}$$

$$A_{c,eff} = b_w \cdot h_{eff} = 300 \cdot 101.95 = 30586.00\ \mathrm{mm}^2$$

$$\rho_{eff} = \frac{A_s}{A_{c,eff}} = \frac{2593.12}{30586.00} = 0.08478$$

$$e_{sm} - e_{cm} = \frac{\sigma_s - k_t \cdot \frac{f_{ct,eff}}{\rho_{p,eff}} \cdot (1 + a_e \cdot \rho_{p,eff})}{E_s} =$$

$$= \frac{0.00 - 0.4 \cdot \frac{2.56}{0.08478} \cdot (1 + 6.35 \cdot 0.08478)}{200000} = -0.00009$$

$$e_{sm} - e_{cm} < 0.6 \cdot \frac{\sigma_s}{E_s} = 0.6 \cdot \frac{0.00}{200000} = 0.00000 \rightarrow e_{sm} - e_{cm} = 0.00000$$

4. Calculation of maximum crack spacing

$$\text{cover } c = 35.635\ \mathrm{mm}$$

$$k_1 = 0.8$$

$$k_2 = 0.5$$

$$s_{r,max} = 3.4 \cdot c \cdot \frac{0.425 \cdot k_1 \cdot k_2 \cdot diameter}{\rho_{p,eff}} = 3.4 \cdot 35.635 \cdot \frac{0.425 \cdot 0.8 \cdot 0.5 \cdot 28.73}{0.08478} = 178.77\ \mathrm{mm}$$

$$s_{r,max} \leq 5 \cdot (c + 0.5 \cdot diameter) = 5 \cdot (35.635 + 0.5 \cdot 28.73) = 250.00\ \mathrm{mm}$$

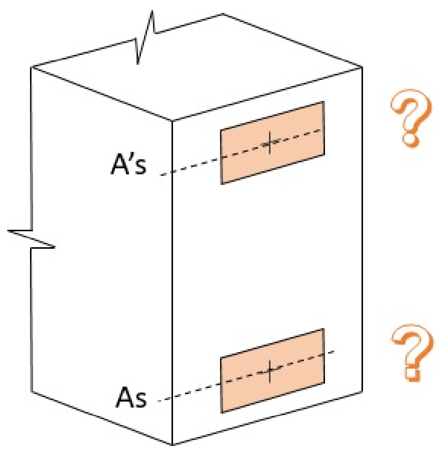
5. Calculation of crack width

$$w_k = (e_{sm} - e_{cm}) \cdot s_{r,max} = 0.00000 \cdot 178.77 = 0.00\ \mathrm{mm}$$

$$w_k = 0.00\ \mathrm{mm} \leq w_{lim} = 0.3\ \mathrm{mm}$$

STATUS OK!

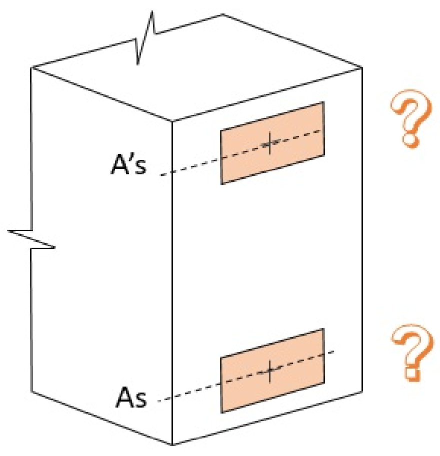
Flexure check (Negative bending moment case)



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!

6.2.3(4)
6.2.3(3)
9.2.2(5)
9.2.2(6)

Shear check

SHEAR FORCE CAPACITY (Members with shear reinforcement)

Section input data:
Design compressive strength of concrete $f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 0.85 \cdot 25 / 1.5 = 14.17\ \mathrm{MPa}$
Design strength of shear rebar $f_{ywd} = f_{ywk} / \gamma_s = 500 / 1.15 = 434.78\ \mathrm{MPa}$
Mean width of web $b_w = 300\ \mathrm{mm}$
Section concrete area $A_c = 202500\ \mathrm{mm}^2$
Tensioning rebar area $A_{sl} = 2593.12\ \mathrm{mm}^2$
Cross-sectional area of the shear reinforcement $A_{sw} = 157.08\ \mathrm{mm}^2$
Spacing of stirrups $s = 250.00\ \mathrm{mm}$
Working depth of reinforcement $d = 550.00\ \mathrm{mm}$
Angle between strut and the beam axis $\theta = 45\ \mathrm{deg}$
Given shear force $V_{Ed} = 0.00\ \mathrm{kN}$

1. Calculation of shear resistance with reinforcement (6.2.3(3),6.2.3(4))

$$\sigma_{cp} = 0 \text{ as } N_{ed} = 0$$

Coefficient taking account of the state of the stress in the compression chord:

$$a_{cw} = 1.0 \text{ as } \sigma_{cp} \leq 0.0\ \mathrm{MPa}$$

Strength reduction factor for concrete cracked in shear

$$v_1 = 0.6 \cdot (1 - \frac{f_{ck}}{250}) = 0.6 \cdot (1 - \frac{25}{250}) = 0.54 \text{ as } f_{ywd} = 434.78\ \mathrm{MPa} \geq 0.8 \cdot f_{ywk} = 400.00\ \mathrm{MPa}$$

$$V_{Rd,max} = \frac{f_{cd} \cdot 0.9 \cdot d \cdot b_w \cdot a_{cw} \cdot v_1}{cot(\theta) + tan(\theta)} = \frac{14.17 \cdot 0.9 \cdot 550.00 \cdot 300 \cdot 1.00 \cdot 0.54}{1.00 + 1.00} = 568.01\ \mathrm{kN}$$

2. Shear Reinforcement detailing (9.2.2(5), 9.2.2(6))
Shear reinforcement ratio

$$\rho_w = \frac{A_{sw}}{b_w \cdot s} = \frac{157.08}{300 \cdot 250} = 0.00209$$

Minimum shear reinforcement ratio

$$\rho_{w,min} = 0.08 \cdot \frac{\sqrt{f_{ck}}}{f_{yk}} = 0.08 \cdot \frac{\sqrt{25}}{500} = 0.00080$$

Minimum vertical links longitudinal spacing

$$s_{max} = 0.75 \cdot d = 0.75 \cdot 550.00 = 412.50\ \mathrm{mm}$$

	Maximum shear reinforcement $$A_{sw,max} = \frac{0.5 \cdot a_{cw} \cdot v_1 \cdot f_{cd} \cdot b_w \cdot s}{f_{ywd}} = \frac{0.5 \cdot 1.00 \cdot 0.54 \cdot 14.17 \cdot 300 \cdot 250}{434.78} = 659.81 \text{ mm}^2$$ $$V_{Ed} = 0.00 \text{ kN} \leq V_{Rd,max} = 568.01 \text{ kN}$$	**STATUS OK!**
	$$\rho_w = 0.00209 \geq \rho_{w,min} = 0.00080$$	**STATUS OK!**
	$$A_{sw} = 157.08 \text{ mm}^2 \leq A_{sw,max} = 659.81 \text{ mm}^2$$	**STATUS OK!**
	$$s = 250.00 \text{ mm} \leq s_{max} = 412.50 \text{ mm}$$	**STATUS OK!**