REFERENCES	CALCULATIONS	RESULTS

Code: ENV 1992-1-1 :1991

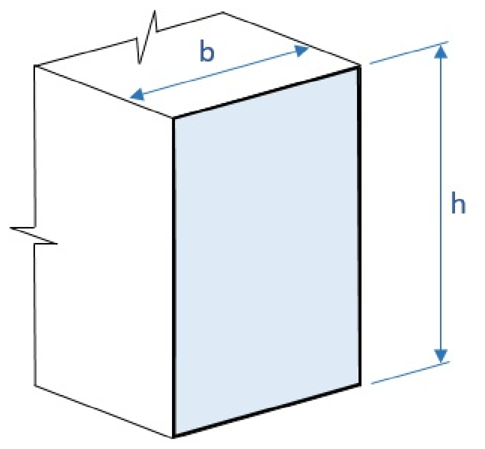
MEMBER #1 (SECTION POSITION 5000.0 mm) BEAM DESIGN REPORT

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 $h = 500 \text{ mm}$
Width $b = 450 \text{ mm}$
Member length = 20000 mm

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

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:
LC 1: 1.35DL (M = 168.75 kN-m, V = 33.75 kN)
LC 2: 1.35DL+1.5LL (M = 244.92 kN-m, V = 48.98 kN)
LC 3: 1.0DL+1.5WL (M = 125.00 kN-m, V = 25.00 kN)
LC 4: 1.35DL+1.5WL (M = 168.75 kN-m, V = 33.75 kN)
LC 5: 1.35DL+1.5LL+0.9WL (M = 244.92 kN-m, V = 48.98 kN)
LC 6: 1.35DL+1.05LL+1.5WL (M = 222.07 kN-m, V = 44.41 kN)
LC 7: 1.0DL+1.0EL (M = 125.00 kN-m, V = 25.00 kN)
LC 8: 1.0DL+0.3LL+1.0EL (M = 140.23 kN-m, V = 28.05 kN)

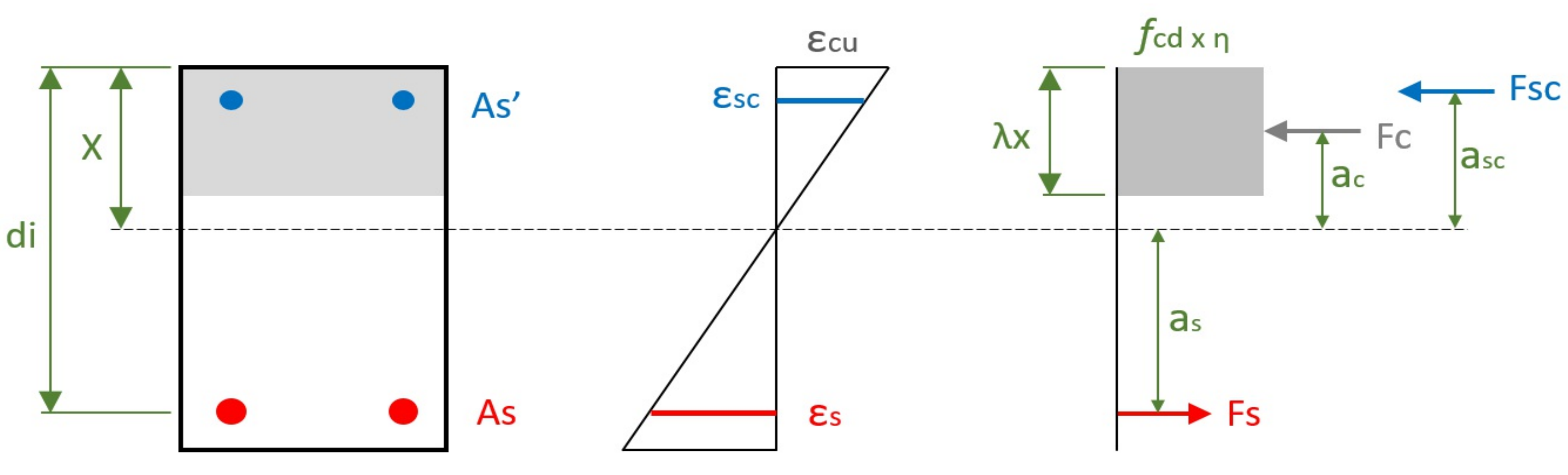
Serviceabiity Limit State:
LC 1: 1.0DL (M = 125.00 kN-m)
LC 2: 1.0DL+1.0LL (M = 175.78 kN-m)

Accepted forces for section check:
Positive moment strength case : (M = 244.92 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 = (244.92 kN-m, V = 48.98 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)

BENDING MOMENT CAPACITY



Section input data:
Section concrete area $A_c = 225000 \text{ mm}^2$
Design compressive strength of concrete $f_{cd} = \alpha_{cc} \cdot f_{ck}/\gamma_c = 0.85 \cdot 25/1.5 = 14.17 \text{ MPa}$
Design strength of rebar $f_{yd} = f_{yk}/\gamma_s = 500/1.15 = 434.78 \text{ MPa}$
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 $M_{Ed} = 244.92 \text{ kN-m}$

Section Rebar

Depth di (mm)	bar diameter (mm)	bar area Asi (mm2)
450	25	490.87
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50	16	201.06
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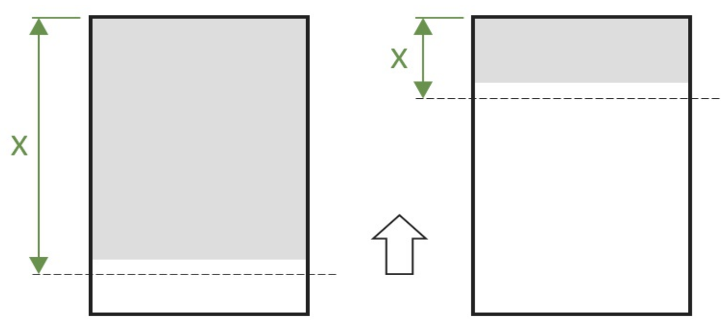
1. Calculation of neutral axis depth x



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 $f_s = \{e_s E_s \ (e_s \le e_y),\ e_y \ (e_s > e_y)\}$
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 450 to 0 mm)

Iter.	x (mm)	As (mm2)	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	450.0	0.0	2295.00	174.83	2469.83	0.00	Infinity
2	441.0	1963.5	2249.10	174.83	2423.93	28.05	86.416
3	432.0	1963.5	2203.20	174.83	2378.03	57.27	41.525
4	423.0	1963.5	2157.30	174.83	2332.13	87.73	26.583
5	414.0	1963.5	2111.40	174.83	2286.23	119.52	19.129
6	405.0	1963.5	2065.50	174.83	2240.33	152.72	14.670
7	396.0	1963.5	2019.60	174.83	2194.43	187.42	11.708
8	387.0	1963.5	1973.70	174.83	2148.53	223.75	9.603
9	378.0	1963.5	1927.80	174.83	2102.63	261.80	8.032
10	369.0	1963.5	1881.90	174.83	2056.73	301.71	6.817
11	360.0	1963.5	1836.00	174.83	2010.83	343.61	5.852
12	351.0	1963.5	1790.10	174.83	1964.93	387.66	5.069

13	342.0	1963.5	1744.20	174.83	1919.03	434.03	4.421
14	333.0	1963.5	1698.30	174.83	1873.13	482.91	3.879
15	324.0	1963.5	1652.40	174.83	1827.23	534.50	3.419
16	315.0	1963.5	1606.50	174.83	1781.33	589.04	3.024
17	306.0	1963.5	1560.60	174.83	1735.43	646.79	2.683
18	297.0	1963.5	1514.70	174.83	1689.53	708.04	2.386
19	288.0	1963.5	1468.80	174.83	1643.63	773.12	2.126
20	279.0	1963.5	1422.90	174.83	1597.73	842.40	1.897
21	270.0	1963.5	1377.00	174.83	1551.83	853.69	1.818
22	261.0	1963.5	1331.10	174.83	1505.93	853.69	1.764
23	252.0	1963.5	1285.20	174.83	1460.03	853.69	1.710
24	243.0	1963.5	1239.30	174.83	1414.13	853.69	1.657
25	234.0	1963.5	1193.40	174.83	1368.23	853.69	1.603
26	225.0	1963.5	1147.50	174.83	1322.33	853.69	1.549
27	216.0	1963.5	1101.60	174.83	1276.43	853.69	1.495
28	207.0	1963.5	1055.70	174.83	1230.53	853.69	1.441
29	198.0	1963.5	1009.80	174.83	1184.63	853.69	1.388
30	189.0	1963.5	963.90	174.83	1138.73	853.69	1.334
31	180.0	1963.5	918.00	174.83	1092.83	853.69	1.280
32	171.0	1963.5	872.10	174.83	1046.93	853.69	1.226
33	162.0	1963.5	826.20	174.83	1001.03	853.69	1.173
34	153.0	1963.5	780.30	174.83	955.13	853.69	1.119
35	144.0	1963.5	734.40	174.83	909.23	853.69	1.065
36	135.0	1963.5	688.50	174.83	863.33	853.69	1.011
(Fc + Fcs) < Fs. Updating of iterations							
1	126.0	1963.5	642.60	169.78	812.38	853.69	0.952
2	134.8	1963.5	687.58	174.83	862.42	853.69	1.010
3	134.6	1963.5	686.66	174.83	861.50	853.69	1.009
4	134.5	1963.5	685.75	174.83	860.58	853.69	1.008
5	134.3	1963.5	684.83	174.83	859.66	853.69	1.007
6	134.1	1963.5	683.91	174.83	858.74	853.69	1.006
7	133.9	1963.5	682.99	174.83	857.83	853.69	1.005
8	133.7	1963.5	682.07	174.83	856.91	853.69	1.004
9	133.6	1963.5	681.16	174.83	855.99	853.69	1.003
10	133.4	1963.5	680.24	174.83	855.07	853.69	1.002
11	133.2	1963.5	679.32	174.83	854.15	853.69	1.001
12	133.0	1963.5	678.40	174.83	853.24	853.69	0.999

Final value of x is 133.02 mm and tensioning rebar area is 1963.48 mm2
Working depth of reinforcement $d = 450.00\ \mathrm{mm}$

2. Calculation moment resistance M_{Rd}

$$M_{Rd} = F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s = 54.14 + 14.51 + 270.60 = 339.26\ \text{kN-m}$$

$$M_{Ed} = 244.92\ \text{kN-m} \leq M_{Rd} = 339.26\ \text{kN-m}$$

STATUS OK!

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

9.2.1.1 (3)
9.2.1.1(1)

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

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

$$A_{s,max} = 0.04 \cdot A_c = 0.04 \cdot 225000 = 9000 \text{ mm}^2$$

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

$$A_{s,min1} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d = 0.26 \cdot \frac{2.56}{500} \cdot 450 \cdot 450.00 = 270.09 \text{ mm}^2$$

$$A_{s,min2} = 0.0013 \cdot b_t \cdot d = 0.0013 \cdot 450 \cdot 450.00 = 303.750 \text{ mm}^2$$

$$A_{s,min} = max\left[A_{s,min1}, A_{s,min2}\right] = 303.75 \text{ mm}^2$$

Check of allowed longitudinal reinforcement

$$A_s = 1963.48 \text{ mm}^2 \leq A_{s,max} = 9000.00 \text{ mm}^2$$

STATUS OK!

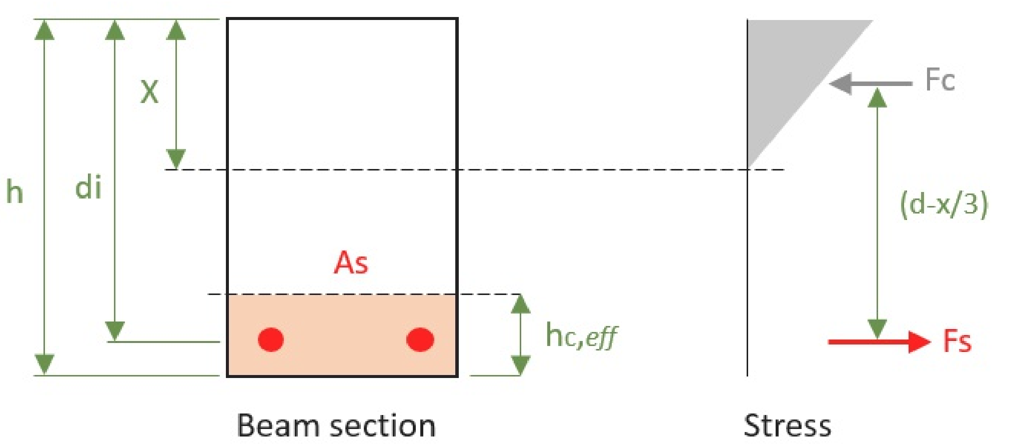
$$A_s = 1963.48 \text{ mm}^2 \geq A_{s,min} = 303.75 \text{ mm}^2$$

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 $d_t = 450 \text{ mm}$
Section concrete area $A_c = 225000 \text{ mm}^2$
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 $M = 175.78$ 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

$$h_0 = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 225000.00}{1900} = 236.84 \text{ mm}$$

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

$$f_{cm} \leq 35\text{MPa} \rightarrow \phi_{RH} = 1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h_0}} = 1 + \frac{1 - \frac{70}{100}}{0.1 \cdot \sqrt[3]{236.84}} = 1.48$$

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

$$\phi_0 = \phi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) = 1.48 \cdot 2.92 \cdot 0.74 = 3.23$$

Coefficient depending on the relative humidity and the notional member size

$$\beta_H = 1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h_0 + 250 = 1.5 \cdot \left[1 + (0.012 \cdot 70)^{18}\right] \cdot 236.84 + 250 = 620.67$$

$$\beta_H \leq 1500$$

Coefficient to describe the development of creep with time after loading

$$\beta_c(t, t_0) = \left[\frac{(t - t_0)}{(\beta_H + t - t_0)}\right]^{0.3} = \left[\frac{(10000 - 3)}{(620.67 + 10000 - 3)}\right]^{0.3} = 0.98$$

Creep coefficient

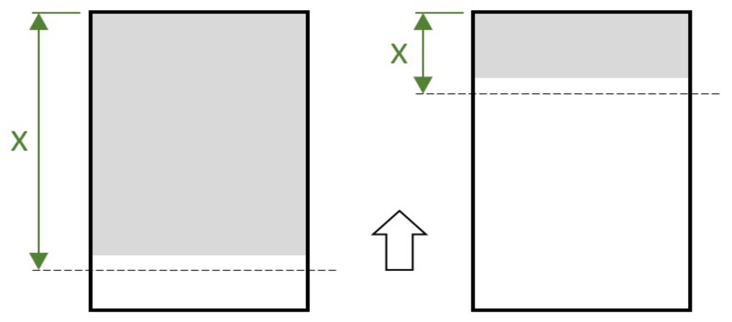
$$\phi(t, t_0) = \phi_0 \cdot \beta_c(t, t_0) = 3.23 \cdot 0.98 = 3.17$$

Effective modulus

$$E_{eff} = \frac{E_{cm}}{1 + \phi(t, t_0)} = \frac{31475.81}{1 + 3.17} = 7549.76 \text{ MPa}$$

$$a_e = \frac{E_s}{E_{eff}} = \frac{200000}{7549.76} = 26.49$$

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 $A_{comp.} + a_e \cdot A_s + a_e \cdot \acute{A_s}$

$\text{Searching of neutral axis x (from 450 to 0 mm)}$

Iter.	x (mm)	As (mm2)	Asc (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	450.00	0.00	402.12	69501.59	119325.10	1.717
2	441.00	1963.48	402.12	67697.32	115152.55	1.701
3	432.00	1963.48	402.12	65929.49	111052.90	1.684
4	423.00	1963.48	402.12	64198.12	107026.15	1.667
5	414.00	1963.48	402.12	62503.19	103072.30	1.649
6	405.00	1963.48	402.12	60844.72	99191.34	1.630
7	396.00	1963.48	402.12	59222.69	95383.29	1.611
8	387.00	1963.48	402.12	57637.12	91648.14	1.590
9	378.00	1963.48	402.12	56087.99	87985.89	1.569
10	369.00	1963.48	402.12	54575.32	84396.54	1.546
11	360.00	1963.48	402.12	53099.09	80880.08	1.523
12	351.00	1963.48	402.12	51659.32	77436.53	1.499
13	342.00	1963.48	402.12	50255.99	74065.88	1.474
14	333.00	1963.48	402.12	48889.12	70768.13	1.448
15	324.00	1963.48	402.12	47558.69	67543.28	1.420
16	315.00	1963.48	402.12	46264.72	64391.32	1.392
17	306.00	1963.48	402.12	45007.19	61312.27	1.362
18	297.00	1963.48	402.12	43786.12	58306.12	1.332
19	288.00	1963.48	402.12	42601.49	55372.87	1.300
20	279.00	1963.48	402.12	41453.32	52512.51	1.267
21	270.00	1963.48	402.12	40341.59	49725.06	1.233
22	261.00	1963.48	402.12	39266.32	47010.51	1.197
23	252.00	1963.48	402.12	38227.49	44368.86	1.161
24	243.00	1963.48	402.12	37225.12	41800.11	1.123
25	234.00	1963.48	402.12	36259.19	39304.25	1.084
26	225.00	1963.48	402.12	35329.72	36881.30	1.044
27	216.00	1963.48	402.12	34436.69	34531.25	1.003
left part < right part. Updating of iterations						
1	207.00	1963.48	402.12	33580.12	32254.10	0.961
2	215.82	1963.48	402.12	34419.21	34484.99	1.002
3	215.64	1963.48	402.12	34401.73	34438.76	1.001
4	215.46	1963.48	402.12	34384.27	34392.57	1.000
5	215.28	1963.48	402.12	34366.83	34346.40	1.00

Value of x is 215.28 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 $d = 450.00\ \mathrm{mm}$

3. Calculation of effective strain

$$\sigma_s = \frac{M}{A_s \cdot (d - x/3)} = \frac{175780000}{1963.48 \cdot (450 - 215.28/3)} = 236.69\ \mathrm{MPa}$$

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

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

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

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

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

$$h_{eff} = max\left[2.5 \cdot (h-d), \frac{h-x}{3}, \frac{h}{2}\right] = max\,[125.00, 94.91, 250.00] = 94.91 \text{ mm}$$

$$A_{c,eff} = b_w \cdot h_{eff} = 450 \cdot 94.91 = 42708.00 \text{ mm}^2$$

$$\rho_{eff} = \frac{A_s}{A_{c,eff}} = \frac{1963.48}{42708.00} = 0.04597$$

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

$$= \frac{236.69 - 0.4 \cdot \frac{2.56}{0.04597} \cdot (1 + 6.35 \cdot 0.04597)}{200000} = 0.00104$$

$$e_{sm} - e_{cm} \geq 0.6 \cdot \frac{\sigma_s}{E_s} = 0.6 \cdot \frac{236.69}{200000} = 0.00071$$

4. Calculation of maximum crack spacing

$$\text{cover } c = 37.5 \text{ mm}$$

$$k_1 = 0.8$$

$$k_2 = 0.5$$

$$s_{r,max} = 3.4 \cdot c \cdot \frac{0.425 \cdot k_1 \cdot k_2 \cdot diameter}{\rho_{p,eff}} = 3.4 \cdot 37.5 \cdot \frac{0.425 \cdot 0.8 \cdot 0.5 \cdot 25}{0.04597} = 219.94 \text{ mm}$$

$$s_{r,max} \leq 5 \cdot (c + 0.5 \cdot diameter) = 5 \cdot (37.5 + 0.5 \cdot 25) = 250.00 \text{ mm}$$

5. Calculation of crack width

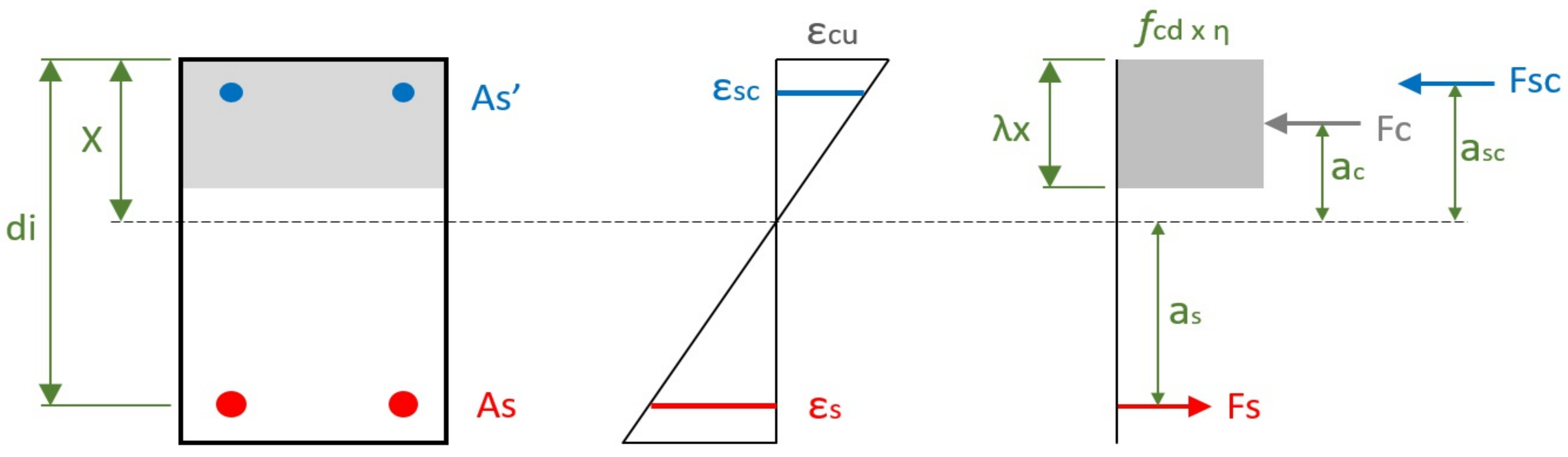
$$w_k = (e_{sm} - e_{cm}) \cdot s_{r,max} = 0.00104 \cdot 219.94 = 0.23 \text{ mm}$$

$$w_k = 0.23 \text{ mm} \leq w_{lim} = 0.3 \text{ mm}$$

STATUS OK!

Flexure check (Negative bending moment case)

BENDING MOMENT CAPACITY



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Design yield strain of rebar $e_y = f_{yd}/E_s = 434.78/200000 = 0.00217$
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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 $M_{Ed} = 0.00\ \mathrm{kN\text{-}m}$

Section Rebar

Depth di (mm)	bar diameter (mm)	bar area Asi (mm2)
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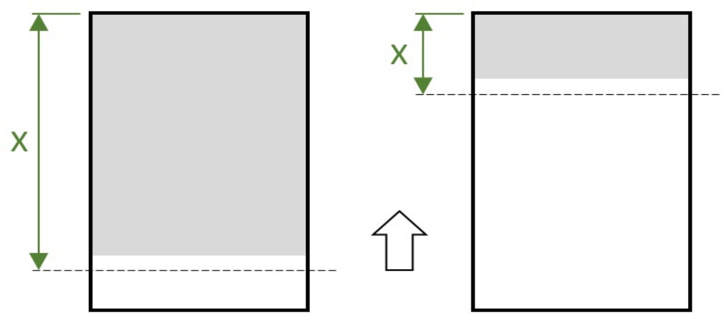

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- Check equilibrium $F_c + F_{cs} = F_s$

Reinforcement stresses $f_s = \{e_s E_s\ (e_s \le e_y),\ e_y\ (e_s > e_y)\}$
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 450 to 0 mm)

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5	414.0	402.1	2111.40	853.69	2965.09	24.48	121.138
6	405.0	402.1	2065.50	853.69	2919.19	31.28	93.336
7	396.0	402.1	2019.60	853.69	2873.29	38.38	74.856
8	387.0	402.1	1973.70	853.69	2827.39	45.82	61.702
9	378.0	402.1	1927.80	853.69	2781.49	53.62	51.878
10	369.0	402.1	1881.90	853.69	2735.59	61.79	44.273
11	360.0	402.1	1836.00	853.69	2689.69	70.37	38.222
12	351.0	402.1	1790.10	853.69	2643.79	79.39	33.300

13	342.0	402.1	1744.20	853.69	2597.89	88.89	29.226
14	333.0	402.1	1698.30	853.69	2551.99	98.90	25.804
15	324.0	402.1	1652.40	853.69	2506.09	109.47	22.894
16	315.0	402.1	1606.50	853.69	2460.19	120.64	20.393
17	306.0	402.1	1560.60	853.69	2414.29	132.46	18.226
18	297.0	402.1	1514.70	853.69	2368.39	145.01	16.333
19	288.0	402.1	1468.80	853.69	2322.49	158.33	14.668
20	279.0	402.1	1422.90	853.69	2276.59	172.52	13.196
21	270.0	402.1	1377.00	853.69	2230.69	174.83	12.759
22	261.0	402.1	1331.10	853.69	2184.79	174.83	12.496
23	252.0	402.1	1285.20	853.69	2138.89	174.83	12.234
24	243.0	402.1	1239.30	853.69	2092.99	174.83	11.971
25	234.0	402.1	1193.40	853.69	2047.09	174.83	11.709
26	225.0	402.1	1147.50	853.69	2001.19	174.83	11.446
27	216.0	402.1	1101.60	853.69	1955.29	174.83	11.184
28	207.0	402.1	1055.70	853.69	1909.39	174.83	10.921
29	198.0	402.1	1009.80	853.69	1863.49	174.83	10.659
30	189.0	402.1	963.90	853.69	1817.59	174.83	10.396
31	180.0	402.1	918.00	853.69	1771.69	174.83	10.133
32	171.0	402.1	872.10	853.69	1725.79	174.83	9.871
33	162.0	402.1	826.20	853.69	1679.89	174.83	9.608
34	153.0	402.1	780.30	853.69	1633.99	174.83	9.346
35	144.0	402.1	734.40	853.69	1588.09	174.83	9.083
36	135.0	402.1	688.50	853.69	1542.19	174.83	8.821
37	126.0	402.1	642.60	829.02	1471.62	174.83	8.417
38	117.0	402.1	596.70	787.07	1383.77	174.83	7.915
39	108.0	402.1	550.80	738.12	1288.92	174.83	7.372
40	99.0	402.1	504.90	680.28	1185.18	174.83	6.779
41	90.0	402.1	459.00	610.86	1069.86	174.83	6.119
42	81.0	402.1	413.10	526.02	939.12	174.83	5.371
43	72.0	402.1	367.20	419.97	787.17	174.83	4.502
44	63.0	402.1	321.30	283.61	604.91	174.83	3.460
45	54.0	402.1	275.40	101.81	377.21	174.83	2.158
(Fc + Fcs) < Fs. Updating of iterations							
1	45.0	2365.6	229.50	0.00	229.50	327.55	0.701
2	53.8	402.1	274.48	97.55	372.04	174.83	2.128
3	53.6	402.1	273.56	93.27	366.83	174.83	2.098
4	53.5	402.1	272.65	88.96	361.60	174.83	2.068
5	53.3	402.1	271.73	84.61	356.34	174.83	2.038
6	53.1	402.1	270.81	80.24	351.05	174.83	2.008
7	52.9	402.1	269.89	75.84	345.73	174.83	1.977
8	52.7	402.1	268.97	71.41	340.38	174.83	1.947
9	52.6	402.1	268.06	66.94	335.00	174.83	1.916
10	52.4	402.1	267.14	62.45	329.59	174.83	1.885

11	52.2	402.1	266.22	57.93	324.15	174.83	1.854
12	52.0	402.1	265.30	53.37	318.67	174.83	1.823
13	51.8	402.1	264.38	48.78	313.17	174.83	1.791
14	51.7	402.1	263.47	44.16	307.63	174.83	1.760
15	51.5	402.1	262.55	39.51	302.06	174.83	1.728
16	51.3	402.1	261.63	34.83	296.46	174.83	1.696
17	51.1	402.1	260.71	30.11	290.82	174.83	1.663
18	50.9	402.1	259.79	25.36	285.16	174.83	1.631
19	50.8	402.1	258.88	20.58	279.45	174.83	1.598
20	50.6	402.1	257.96	15.76	273.72	174.83	1.566
21	50.4	402.1	257.04	10.91	267.95	174.83	1.533
22	50.2	402.1	256.12	6.02	262.14	174.83	1.499
23	50.0	402.1	255.20	1.10	256.30	174.83	1.466
24	49.9	2365.6	254.29	0.00	254.29	178.69	1.423
25	49.7	2365.6	253.37	0.00	253.37	183.69	1.379
26	49.5	2365.6	252.45	0.00	252.45	188.72	1.338
27	49.3	2365.6	251.53	0.00	251.53	193.78	1.298
28	49.1	2365.6	250.61	0.00	250.61	198.89	1.260
29	49.0	2365.6	249.70	0.00	249.70	204.03	1.224
30	48.8	2365.6	248.78	0.00	248.78	209.21	1.189
31	48.6	2365.6	247.86	0.00	247.86	214.43	1.156
32	48.4	2365.6	246.94	0.00	246.94	219.68	1.124
33	48.2	2365.6	246.02	0.00	246.02	224.98	1.094
34	48.1	2365.6	245.11	0.00	245.11	230.32	1.064
35	47.9	2365.6	244.19	0.00	244.19	235.69	1.036
36	47.7	2365.6	243.27	0.00	243.27	241.11	1.009
37	47.5	2365.6	242.35	0.00	242.35	246.56	0.983

Final value of x is 47.52 mm and tensioning rebar area is 2365.60 mm2
Working depth of reinforcement $d = 117.99\ \mathrm{mm}$

STATUS OK!

2. Calculation moment resistance M_{Rd}

$$M_{Rd} = F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s = 6.91 + 0.00 + 70.55 = 77.46\ \text{kN-m}$$

$$M_{Ed} = 0.00\ \text{kN-m} \leq M_{Rd} = 77.46\ \text{kN-m}$$

9.2.1.1 (3)
9.2.1.1(1)

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

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

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

$$A_{s,max} = 0.04 \cdot A_c = 0.04 \cdot 225000 = 9000\ \mathrm{mm}^2$$

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

$$A_{s,min1} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d = 0.26 \cdot \frac{2.56}{500} \cdot 450 \cdot 117.99 = 70.82\ \mathrm{mm}^2$$

$$A_{s,min2} = 0.0013 \cdot b_t \cdot d = 0.0013 \cdot 450 \cdot 117.99 = 79.643\ \mathrm{mm}^2$$

$$A_{s,min} = max\left[A_{s,min1}, A_{s,min2}\right] = 79.64\ \mathrm{mm}^2$$

Check of allowed longitudinal reinforcement

$$A_s = 2365.60\ \mathrm{mm}^2 \leq A_{s,max} = 9000.00\ \mathrm{mm}^2$$

STATUS OK!

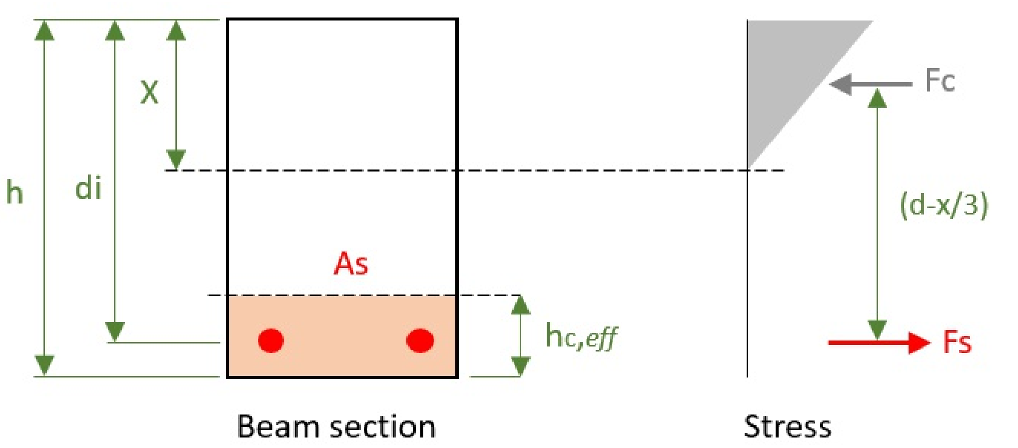
$$A_s = 2365.60\ \mathrm{mm}^2 \geq A_{s,min} = 79.64\ \mathrm{mm}^2$$

STATUS OK!

Crack width check (Negative bending moment case)

7.3.4(2)

CRACK WIDTH CAPACITY



Section input data:
Depth to the outermost tension side of reinforcement $d_t = 450\ \mathrm{mm}$
Section concrete area $A_c = 225000\ \mathrm{mm}^2$
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 $M = 0.00\ \mathrm{kN\text{-}m}$

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

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

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

Notional size of the member

$$h_0 = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 225000.00}{1900} = 236.84\ \mathrm{mm}$$

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

$$f_{cm} \leq 35\mathrm{MPa} \rightarrow \phi_{RH} = 1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h_0}} = 1 + \frac{1 - \frac{70}{100}}{0.1 \cdot \sqrt[3]{236.84}} = 1.48$$

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

$$\phi_0 = \phi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) = 1.48 \cdot 2.92 \cdot 0.74 = 3.23$$

Coefficient depending on the relative humidity and the notional member size

$$\beta_H = 1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h_0 + 250 = 1.5 \cdot \left[1 + (0.012 \cdot 70)^{18}\right] \cdot 236.84 + 250 = 620.67$$

$$\beta_H \leq 1500$$

Coefficient to describe the development of creep with time after loading

$$\beta_c(t, t_0) = \left[\frac{(t - t_0)}{(\beta_H + t - t_0)}\right]^{0.3} = \left[\frac{(10000 - 3)}{(620.67 + 10000 - 3)}\right]^{0.3} = 0.98$$

Creep coefficient

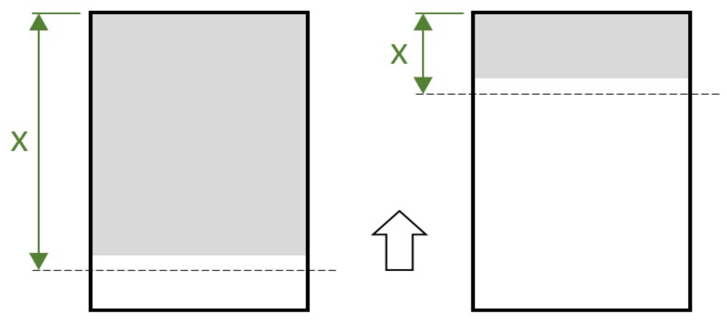
$$\phi(t,t_0) = \phi_0 \cdot \beta_c(t,t_0) = 3.23 \cdot 0.98 = 3.17$$

Effective modulus

$$E_{eff} = \frac{E_{cm}}{1+\phi(t,t_0)} = \frac{31475.81}{1+3.17} = 7549.76\ \text{MPa}$$

$$a_e = \frac{E_s}{E_{eff}} = \frac{200000}{7549.76} = 26.49$$

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 450 to 0 mm)

Iter.	x (mm)	As (mm2)	Asc (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	450.00	0.00	1963.48	52956.86	119325.10	2.253
2	441.00	402.12	1963.48	51152.58	115152.55	2.251
3	432.00	402.12	1963.48	49384.76	111052.90	2.249
4	423.00	402.12	1963.48	47653.38	107026.15	2.246
5	414.00	402.12	1963.48	45958.46	103072.30	2.243
6	405.00	402.12	1963.48	44299.98	99191.34	2.239
7	396.00	402.12	1963.48	42677.96	95383.29	2.235
8	387.00	402.12	1963.48	41092.38	91648.14	2.230
9	378.00	402.12	1963.48	39543.26	87985.89	2.225
10	369.00	402.12	1963.48	38030.58	84396.54	2.219
11	360.00	402.12	1963.48	36554.36	80880.08	2.213
12	351.00	402.12	1963.48	35114.58	77436.53	2.205
13	342.00	402.12	1963.48	33711.26	74065.88	2.197
14	333.00	402.12	1963.48	32344.38	70768.13	2.188
15	324.00	402.12	1963.48	31013.96	67543.28	2.178
16	315.00	402.12	1963.48	29719.98	64391.32	2.167
17	306.00	402.12	1963.48	28462.46	61312.27	2.154
18	297.00	402.12	1963.48	27241.38	58306.12	2.140
19	288.00	402.12	1963.48	26056.76	55372.87	2.125
20	279.00	402.12	1963.48	24908.58	52512.51	2.108
21	270.00	402.12	1963.48	23796.86	49725.06	2.090
22	261.00	402.12	1963.48	22721.58	47010.51	2.069
23	252.00	402.12	1963.48	21682.76	44368.86	2.046
24	243.00	402.12	1963.48	20680.38	41800.11	2.021
25	234.00	402.12	1963.48	19714.46	39304.25	1.994
26	225.00	402.12	1963.48	18784.98	36881.30	1.963
27	216.00	402.12	1963.48	17891.96	34531.25	1.930

28	207.00	402.12	1963.48	17035.38	32254.10	1.893
29	198.00	402.12	1963.48	16215.26	30049.85	1.853
30	189.00	402.12	1963.48	15431.58	27918.49	1.809
31	180.00	402.12	1963.48	14684.36	25860.04	1.761
32	171.00	402.12	1963.48	13973.58	23874.49	1.709
33	162.00	402.12	1963.48	13299.26	21961.84	1.651
34	153.00	402.12	1963.48	12661.38	20122.09	1.589
35	144.00	402.12	1963.48	12059.96	18355.23	1.522
36	135.00	402.12	1963.48	11494.98	16661.28	1.449
37	126.00	402.12	1963.48	10966.46	15040.23	1.371
38	117.00	402.12	1963.48	10474.38	13492.08	1.288
39	108.00	402.12	1963.48	10018.76	12016.83	1.199
40	99.00	402.12	1963.48	9599.58	10614.47	1.106
41	90.00	402.12	1963.48	9216.86	9285.02	1.007
left part < right part. Updating of iterations						
1	81.00	402.12	1963.48	8870.58	8028.47	0.905
2	89.82	402.12	1963.48	9209.57	9259.18	1.005
3	89.64	402.12	1963.48	9202.30	9233.36	1.003
4	89.46	402.12	1963.48	9195.05	9207.57	1.001
5	89.28	402.12	1963.48	9187.81	9181.81	1.00

Value of x is 89.28 mm
Tensioning rebar area $A_s = 402.12\ \mathrm{mm}^2$
Compression rebar area $A_{sc} = 1963.48\ \mathrm{mm}^2$
Working depth of reinforcement $d = 450.00\ \mathrm{mm}$

3. Calculation of effective strain

$$\sigma_s = \frac{M}{A_s \cdot (d - x/3)} = \frac{0}{402.12 \cdot (450 - 89.28/3)} = 0.00\ \mathrm{MPa}$$

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

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

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

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

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

$$h_{eff} = max\left[2.5 \cdot (h-d), \frac{h-x}{3}, \frac{h}{2}\right] = max\left[125.00, 136.91, 250.00\right] = 125.00\ \mathrm{mm}$$

$$A_{c,eff} = b_w \cdot h_{eff} = 450 \cdot 125.00 = 56250.00\ \mathrm{mm}^2$$

$$\rho_{eff} = \frac{A_s}{A_{c,eff}} = \frac{402.12}{56250.00} = 0.00715$$

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

$$= \frac{0.00 - 0.4 \cdot \frac{2.56}{0.00715} \cdot (1 + 6.35 \cdot 0.00715)}{200000} = -0.00075$$

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

4. Calculation of maximum crack spacing

$$\text{cover } c = 42 \text{ mm}$$

$$k_1 = 0.8$$

$$k_2 = 0.5$$

$$s_{r,max} = 3.4 \cdot c \cdot \frac{0.425 \cdot k_1 \cdot k_2 \cdot diameter}{\rho_{p,eff}} = 3.4 \cdot 42 \cdot \frac{0.425 \cdot 0.8 \cdot 0.5 \cdot 16}{0.00715} = 523.28 \text{ mm}$$

$$s_{r,max} > 5 \cdot (c + 0.5 \cdot diameter) = 5 \cdot (42 + 0.5 \cdot 16) = 250.00 \text{ mm} \rightarrow s_{r,max} = 250.00$$

5. Calculation of crack width

$$w_k = (e_{sm} - e_{cm}) \cdot s_{r,max} = 0.00000 \cdot 250.00 = 0.00 \text{ mm}$$

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

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 $f_{cd} = \alpha_{cc} \cdot f_{ck}/\gamma_c = 0.85 \cdot 25/1.5 = 14.17 \text{ MPa}$
Design strength of shear rebar $f_{ywd} = f_{ywk}/\gamma_s = 500/1.15 = 434.78 \text{ MPa}$
Mean width of web $b_w = 450 \text{ mm}$
Section concrete area $A_c = 225000 \text{ mm}^2$
Tensioning rebar area $A_{sl} = 1963.48 \text{ mm}^2$
Cross-sectional area of the shear reinforcement $A_{sw} = 157.08 \text{ mm}^2$
Spacing of stirrups $s = 250.00 \text{ mm}$
Working depth of reinforcement $d = 450.00 \text{ mm}$
Angle between strut and the beam axis $\theta = 45 \text{ deg}$
Given shear force $V_{Ed} = 48.98 \text{ kN}$

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:

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

Strength reduction factor for concrete cracked in shear

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

$$V_{Rd,max} = \frac{f_{cd} \cdot 0.9 \cdot d \cdot b_w \cdot a_{cw} \cdot v_1}{cot(\theta) + tan(\theta)} = \frac{14.17 \cdot 0.9 \cdot 450.00 \cdot 450 \cdot 1.00 \cdot 0.54}{1.00 + 1.00} = 697.11 \text{ kN}$$

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

$$\rho_w = \frac{A_{sw}}{b_w \cdot s} = \frac{157.08}{450 \cdot 250} = 0.00140$$

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

$$s_{max} = 0.75 \cdot d = 0.75 \cdot 450.00 = 337.50 \text{ mm}$$

Maximum shear reinforcement

$$A_{sw,max} = \frac{0.5 \cdot a_{cw} \cdot v_1 \cdot f_{cd} \cdot b_w \cdot s}{f_{ywd}} = \frac{0.5 \cdot 1.00 \cdot 0.54 \cdot 14.17 \cdot 450 \cdot 250}{434.78} = 989.72 \text{ mm}^2$$

$$V_{Ed} = 48.98 \text{ kN} \leq V_{Rd,max} = 697.11 \text{ kN}$$

STATUS OK!

$$\rho_w = 0.00140 \geq \rho_{w,min} = 0.00080$$

STATUS OK!

$$A_{sw} = 157.08 \text{ mm}^2 \leq A_{sw,max} = 989.72 \text{ mm}^2$$

STATUS OK!

$$s = 250.00 \text{ mm} \leq s_{max} = 337.50 \text{ mm}$$

STATUS OK!

Deflection check

DEFLECTION OF BEAM (short-term and long-term)

Section input data:
Member span $L = 10000 \text{ mm}$
Load duration factor $\beta = 0.5$
Concrete cement class : N
Deflection limitation $L/250$
Age of concrete at end of curing $t_s = 7 \text{ days}$
Typical dflection factor $k = 0.104$
Given bending moment $M = 175.78 \text{ kN-m}$

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

$$I_{uc} = \frac{b \cdot h^3}{12} = \frac{450 \cdot 500^3}{12} = 4687500000.00 \text{ mm}^4$$

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

$$I_{cr} = \frac{b \cdot x^3}{3} + a_e \cdot A_s \cdot (d - x)^2 = \frac{450 \cdot 215.28^3}{3} + 26.49 \cdot 1963.48 \cdot (450.00 - 215.28)^2$$
$$= 4362241000.51 \text{ mm}^4$$

3. Moment that will cause cracking of the section:

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

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

$$M_{cr} = \frac{f_{ctm} \cdot I_{uc}}{y_t} = \frac{2.56 \cdot 4687500000.00}{250.00} = 48.09 \text{ kN-m}$$

4. Calculate the curvature of uncracked $(1/r)_{uc}$ and cracked $(1/r)_{cr}$ section

$$\xi = 1 - \beta \cdot (M_{cr}/M_s^*)^2 = 1 - 0.5 \cdot (48.09/175.78)^2 = 0.96$$

$$(1/r)_{uc} = \frac{M}{E_{c,} \cdot I_{uc}} = \frac{175.78}{7549.76 \cdot 4687500000.00} = 0.00000497 \text{ /mm}$$

$$(1/r)_{cr} = \frac{M}{E_c \cdot I_{cr}} = \frac{175.78}{7549.76 \cdot 4362241000.51} = 0.00000534 \text{ /mm}$$

$$1/r = \xi \cdot (1/r)_{cr} + (1 - \xi) \cdot (1/r)_{uc} = 0.96 \cdot 0.00000534 + (1 - 0.96) \cdot 0.00000497$$
$$= 0.00000532 \text{ /mm}$$

5. Calculate srinkage curvature

$$f_{cm0} = 10 \text{ MPa}$$

$$RH_0 = 100\ \%$$

$$a_{ds1} = 4$$

$$a_{ds2} = 0.12$$

$$\beta_{RH} = 1.55 \cdot \left[1 - (\frac{RH}{RH_0})^3\right] = 1.55 \cdot \left[1 - (\frac{70}{100})^3\right] = 1.01835$$

Basic drying shrinkage strain (B.11)

$$e_{cd,0} = 0.85 \cdot \left[(220 + 110 \cdot a_{ds1}) \cdot exp(-a_{ds2} \cdot \frac{f_{cm}}{f_{cm0}})\right] \cdot 10^{-6} \cdot \beta_{RH} =$$

$$= 0.85 \cdot \left[(220 + 110 \cdot 4) \cdot exp(-0.12 \cdot \frac{33.00}{10.00})\right] \cdot 10^{-6} \cdot 1.02 = 0.000384$$

Autogenous shrinkage strain (3.12):

$$e_{ca} = 2.5 \cdot (f_{ck} - 10) \cdot 10^{-6} = 2.5 \cdot (25 - 10) \cdot 10^{-6} = 0.000037$$

Notional size of the member

$$h_0 = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 225000.00}{1900} = 236.84\ \mathrm{mm}$$

Time-development coefficients (3.10, 3.13)

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0.04 \cdot \sqrt{h_0^3}} = \frac{(10000 - 7)}{(10000 - 7) + 0.04 \cdot \sqrt{236.84^3}} = 0.98562$$

$$\beta_{as}(t) = 1 - exp(-0.2 \cdot t^{0.5}) = 1 - exp(-0.2 \cdot 10000^{0.5}) = 1.000000$$

$$k_h = 0.81$$

$$e_{ca}(t) = \beta_{as}(t) \cdot e_{ca} = 1.0000 \cdot 0.00004 = 0.00004$$

$$e_{cd}(t) = \beta_{ds}(t, t_s) \cdot k_h \cdot e_{cd,0} = 0.98562 \cdot 0.81 \cdot 0.00038 = 0.00031$$

Final shrinkage strain at infinite time

$$e_{cs} = e_{ca}(t) + e_{cd}(t) = 0.00004 + 0.00031 = 0.00035$$

Shrinkage curvature for cracked section

$$S = A_s \cdot (d - x) = 1963.48 \cdot (450.00 - 215.28) = 460868.03\ \mathrm{mm}^3$$

$$(1/r_{cs})_{cr} = e_{cs} \cdot a_e \cdot S/I_{cr} = 0.000346 \cdot 26.49 \cdot 460868.03/4362241000.51 = 0.00000097\ /\mathrm{mm}$$

Shrinkage curvature for uncracked section

$$S = A_s \cdot (d - 0.5 \cdot h) = 1963.48 \cdot (450.00 - o.5 \cdot 500) = 392696.00\ \mathrm{mm}^3$$

$$(1/r_{cs})_{uc} = e_{cs} \cdot a_e \cdot S/I_{uc} = 0.000346 \cdot 26.49 \cdot 392696.00/4687500000.00 = 0.00000077\ /\mathrm{mm}$$

The average shrinkage curvature

$$(1/r)_{cs} = \xi \cdot (1/r_{cs})_{cr} + (1 - \xi) \cdot (1/r_{cs})_{uc} = 0.96 \cdot 0.00000097 + (1 - 0.96) \cdot 0.00000077 = 0.00000096\ /\mathrm{mm}$$

. Calculate the deflection based on the section curvature

$$\text{Do to loading: } \triangle_1 = k \cdot L^2 \cdot (1/r) = 0.104 \cdot 10000^2 \cdot 0.00000532 = 55.364\ \mathrm{mm}$$

$$\text{Do to loading: } \triangle_2 = k \cdot L^2 \cdot (1/r)_{cs} = 0.104 \cdot 10000^2 \cdot 0.00000096 = 9.983\ \mathrm{mm}$$

$$\text{Total: } \triangle_{total} = \triangle_1 + \triangle_2 = 55.364 + 9.983 = 65.347\ \mathrm{mm}$$

	$\triangle_{total} = 65.35 \text{ mm} > (span/250) = (10000/250) = 40.00 \text{ mm}$	**STATUS NG!**