REFERENCES	CALCULATIONS	RESULTS

MEMBER #1 (SECTION POSITION 4750.0 mm) BEAM DESIGN REPORT

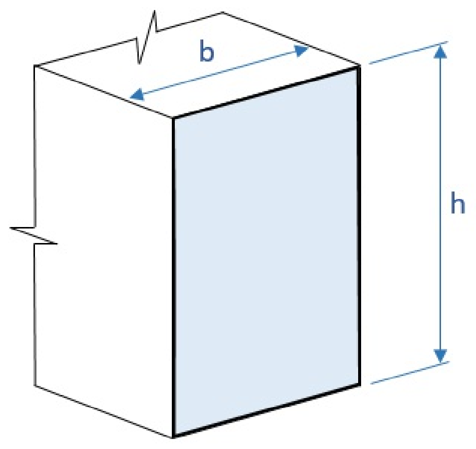
Code: ENV 1992-1-1 :1991

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 $h = 700\ \mathrm{mm}$
Width $b = 300\ \mathrm{mm}$
Member length = $9500\ \mathrm{mm}$

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:
LC 1: null (M = 0.00 kN-m, V = 0.00 kN)

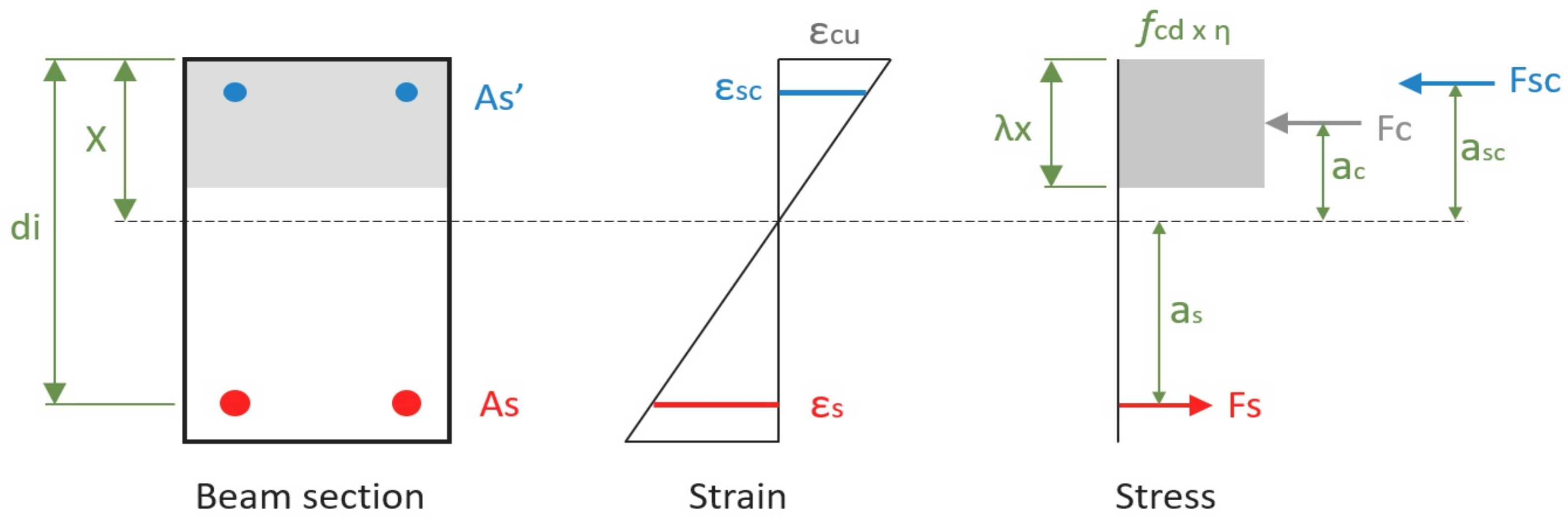
Serviceabiity Limit State:
LC 1: 1.0DL (M = 199.99 kN-m)

Accepted forces for section check:
Positive moment strength case : (M = 0.00 kN-m)
Positive moment service. case: (M = 199.99 kN-m)
Negative moment strength case: (M = 0.00 kN-m)
Negative moment service. case: (M = 0.00 kN-m)
Shear strength case: M = (0.00 kN-m, V = 0.00 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



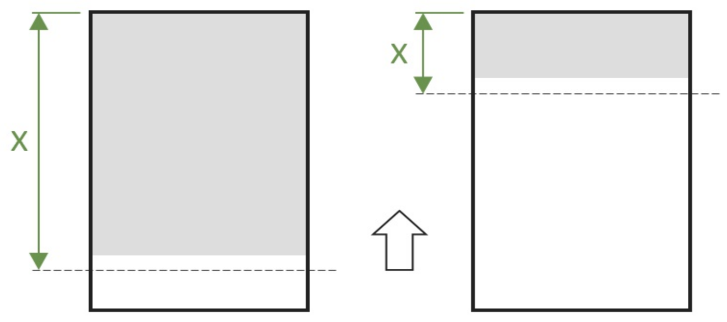
Section input data:

Section concrete area $A_c = 210000\ \mathrm{mm}^2$
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 $M_{Ed} = 0.00\ \mathrm{kN\text{-}m}$

Section Rebar

Depth di (mm)	**bar diameter (mm)**	**bar area Asi (mm2)**
634	25	490.87
634	25	490.87
634	25	490.87
550	25	490.87
550	25	490.87

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 634 to 0 mm)

Iter.	x (mm)	As (mm2)	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	634.0	0.0	2155.60	91.05	2246.65	0.00	Infinity
2	621.3	1472.6	2112.49	78.88	2191.37	21.04	104.166
3	608.6	1472.6	2069.38	66.21	2135.59	42.95	49.721
4	596.0	1472.6	2026.26	53.00	2079.26	65.80	31.601
5	583.3	1472.6	1983.15	39.21	2022.36	89.64	22.562
6	570.6	1472.6	1940.04	24.81	1964.85	114.54	17.155
7	557.9	1472.6	1896.93	9.76	1906.68	140.57	13.564
8	545.2	2454.3	1853.82	0.00	1853.82	173.81	10.666
9	532.6	2454.3	1810.70	0.00	1810.70	218.85	8.274
10	519.9	2454.3	1767.59	0.00	1767.59	266.09	6.643
11	507.2	2454.3	1724.48	0.00	1724.48	315.70	5.462
12	494.5	2454.3	1681.37	0.00	1681.37	367.84	4.571
13	481.8	2454.3	1638.26	0.00	1638.26	422.74	3.875
14	469.2	2454.3	1595.14	0.00	1595.14	480.60	3.319
15	456.5	2454.3	1552.03	0.00	1552.03	541.67	2.865
16	443.8	2454.3	1508.92	0.00	1508.92	606.23	2.489
17	431.1	2454.3	1465.81	0.00	1465.81	674.59	2.173
18	418.4	2454.3	1422.70	0.00	1422.70	747.10	1.904
19	405.8	2454.3	1379.58	0.00	1379.58	824.13	1.674
20	393.1	2454.3	1336.47	0.00	1336.47	906.14	1.475
21	380.4	2454.3	1293.36	0.00	1293.36	946.66	1.366
22	367.7	2454.3	1250.25	0.00	1250.25	980.92	1.275
23	355.0	2454.3	1207.14	0.00	1207.14	1017.63	1.186
24	342.4	2454.3	1164.02	0.00	1164.02	1057.06	1.101
25	329.7	2454.3	1120.91	0.00	1120.91	1067.11	1.050
26	317.0	2454.3	1077.80	0.00	1077.80	1067.11	1.010
(Fc + Fcs) < Fs. Updating of iterations							
1	304.3	2454.3	1034.69	0.00	1034.69	1067.11	0.970
2	316.7	2454.3	1076.94	0.00	1076.94	1067.11	1.009
3	316.5	2454.3	1076.08	0.00	1076.08	1067.11	1.008
4	316.2	2454.3	1075.21	0.00	1075.21	1067.11	1.008
5	316.0	2454.3	1074.35	0.00	1074.35	1067.11	1.007
6	315.7	2454.3	1073.49	0.00	1073.49	1067.11	1.006
7	315.5	2454.3	1072.63	0.00	1072.63	1067.11	1.005
8	315.2	2454.3	1071.76	0.00	1071.76	1067.11	1.004
9	315.0	2454.3	1070.90	0.00	1070.90	1067.11	1.004
10	314.7	2454.3	1070.04	0.00	1070.04	1067.11	1.003
11	314.5	2454.3	1069.18	0.00	1069.18	1067.11	1.002
12	314.2	2454.3	1068.32	0.00	1068.32	1067.11	1.001
13	314.0	2454.3	1067.45	0.00	1067.45	1067.11	1.000
14	313.7	2454.3	1066.59	0.00	1066.59	1067.11	1.000

Final value of x is 313.70 mm and tensioning rebar area is 2454.35 mm2

Working depth of reinforcement $d = 600.40 \text{ mm}$

2. Calculation moment resistance M_{Rd}

$$M_{Rd} = F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s = 200.76 + 0.00 + 305.94 = 506.69 \text{ kN-m}$$

$$M_{Ed} = 0.00 \text{ kN-m} \leq M_{Rd} = 506.69 \text{ kN-m}$$

STATUS OK!

9.2.1.1 (3)
9.2.1.1(1)

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

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

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

$$A_{s,max} = 0.04 \cdot A_c = 0.04 \cdot 210000 = 8400 \text{ mm}^2$$

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

$$A_{s,min1} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b_t \cdot d = 0.26 \cdot \frac{2.56}{500} \cdot 300 \cdot 600.40 = 240.24 \text{ mm}^2$$

$$A_{s,min2} = 0.0013 \cdot b_t \cdot d = 0.0013 \cdot 300 \cdot 600.40 = 270.180 \text{ mm}^2$$

$$A_{s,min} = max\left[A_{s,min1}, A_{s,min2}\right] = 270.18 \text{ mm}^2$$

Check of allowed longitudinal reinforcement

$$A_s = 2454.35 \text{ mm}^2 \leq A_{s,max} = 8400.00 \text{ mm}^2$$

STATUS OK!

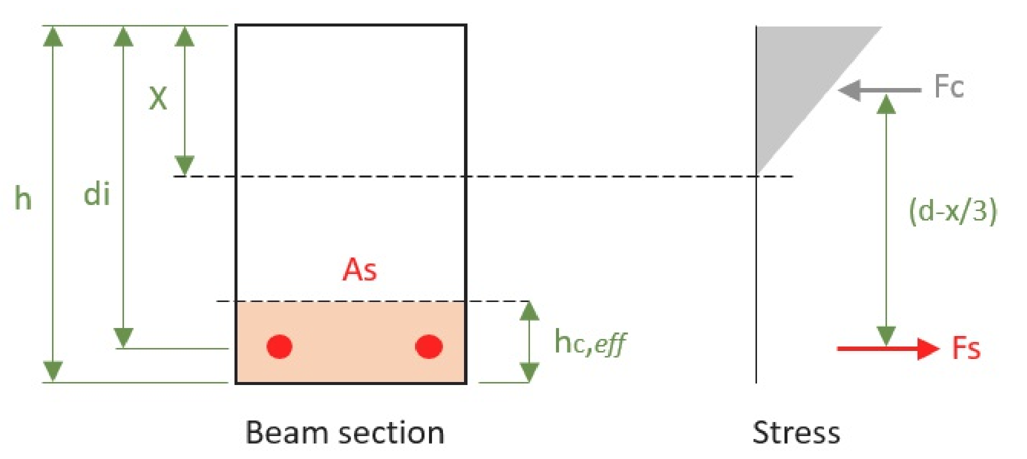
$$A_s = 2454.35 \text{ mm}^2 \geq A_{s,min} = 270.18 \text{ mm}^2$$

STATUS OK!

7.3.4(2)

Crack width check (Positive bending moment case)

CRACK WIDTH CAPACITY



Section input data:
Depth to the outermost tension side of reinforcement $d_t = 634 \text{ mm}$
Section concrete area $A_c = 210000 \text{ mm}^2$
Age of concrete at loading $t_0 = 3 \text{ days}$
Age of concrete at the moment considered $t = 450 \text{ days}$
Relative humidity $RH = 60 \%$
Given bending moment $M = 199.99 \text{ kN-m}$

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

$$h_0 = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 210000.00}{2000} = 210.00 \text{ mm}$$

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

$$f_{cm} \leq 35\text{MPa} \rightarrow \phi_{RH} = 1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h_0}} = 1 + \frac{1 - \frac{60}{100}}{0.1 \cdot \sqrt[3]{210.00}} = 1.67$$

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

$$\phi_0 = \phi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) = 1.67 \cdot 2.92 \cdot 0.74 = 3.64$$

Coefficient depending on the relative humidity and the notional member size

$$\beta_H = 1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h_0 + 250 = 1.5 \cdot \left[1 + (0.012 \cdot 60)^{18}\right] \cdot 210.00 + 250 = 565.85$$

$$\beta_H \leq 1500$$

Coefficient to describe the development of creep with time after loading

$$\beta_c(t, t_0) = \left[\frac{(t - t_0)}{(\beta_H + t - t_0)}\right]^{0.3} = \left[\frac{(450 - 3)}{(565.85 + 450 - 3)}\right]^{0.3} = 0.78$$

Creep coefficient

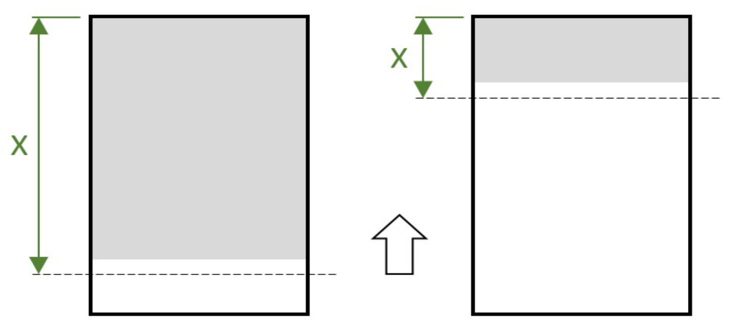
$$\phi(t, t_0) = \phi_0 \cdot \beta_c(t, t_0) = 3.64 \cdot 0.78 = 2.84$$

Effective modulus

$$E_{eff} = \frac{E_{cm}}{1 + \phi(t, t_0)} = \frac{31475.81}{1 + 2.84} = 8187.21 \text{ MPa}$$

$$a_e = \frac{E_s}{E_{eff}} = \frac{200000}{8187.21} = 24.43$$

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 634 to 0 mm)

Iter.	x (mm)	As (mm2)	Asc (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	634.00	0.00	981.74	96290.80	158598.71	1.647
2	621.32	1472.61	981.74	93903.18	153063.23	1.630
3	608.64	1472.61	981.74	91563.79	147624.23	1.612
4	595.96	1472.61	981.74	89272.65	142281.69	1.594
5	583.28	1472.61	981.74	87029.73	137035.62	1.575
6	570.60	1472.61	981.74	84835.05	131886.03	1.555
7	557.92	1472.61	981.74	82688.61	126832.90	1.534
8	545.24	2454.35	0.00	80590.40	121876.24	1.512
9	532.56	2454.35	0.00	78540.42	117016.05	1.490
10	519.88	2454.35	0.00	76538.68	112252.33	1.467
11	507.20	2454.35	0.00	74585.17	107585.08	1.442
12	494.52	2454.35	0.00	72679.90	103014.30	1.417
13	481.84	2454.35	0.00	70822.86	98539.99	1.391
14	469.16	2454.35	0.00	69014.06	94162.14	1.364
15	456.48	2454.35	0.00	67253.50	89880.77	1.336
16	443.80	2454.35	0.00	65541.16	85695.87	1.308
17	431.12	2454.35	0.00	63877.06	81607.43	1.278
18	418.44	2454.35	0.00	62261.20	77615.47	1.247
19	405.76	2454.35	0.00	60693.57	73719.97	1.215
20	393.08	2454.35	0.00	59174.18	69920.95	1.182
21	380.40	2454.35	0.00	57703.02	66218.39	1.148
22	367.72	2454.35	0.00	56280.10	62612.31	1.113
23	355.04	2454.35	0.00	54905.41	59102.69	1.076
24	342.36	2454.35	0.00	53578.95	55689.54	1.039
25	329.68	2454.35	0.00	52300.73	52372.86	1.001
left part < right part. Updating of iterations						
1	317.00	2454.35	0.00	51070.75	49152.65	0.962
2	329.43	2454.35	0.00	52275.66	52307.51	1.001
3	329.17	2454.35	0.00	52250.61	52242.20	1.00

Value of x is 329.17 mm
Tensioning rebar area $A_s = 2454.35\ \mathrm{mm}^2$
Compression rebar area $A_{sc} = 0.00\ \mathrm{mm}^2$
Working depth of reinforcement $d = 600.40\ \mathrm{mm}$

3. Calculation of effective strain

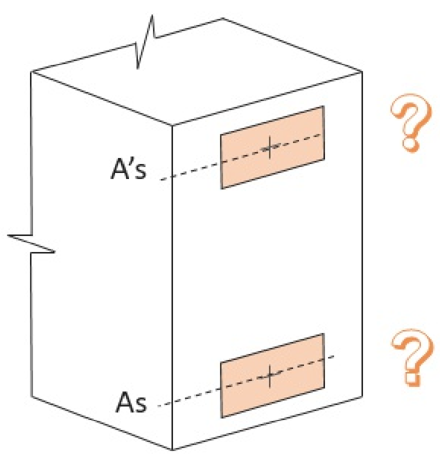
$$\sigma_s = \frac{M}{A_s \cdot (d - x/3)} = \frac{199994000}{2454.35 \cdot (600.4 - 329.17/3)} = 166.07\ \mathrm{MPa}$$

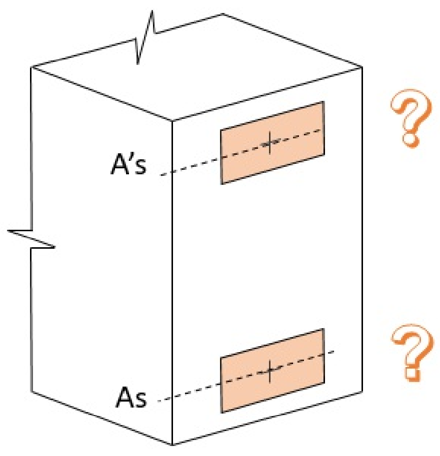
$$k_t = 0.4 \text{ as long term loading}$$

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

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

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

	$$a_e = \frac{E_s}{E_{cm}} = \frac{200000}{31475.81} = 6.35$$ $$h_{eff} = max\left[2.5 \cdot (h-d), \frac{h-x}{3}, \frac{h}{2}\right] = max\,[249.00, 123.61, 350.00] = 123.61\ \mathrm{mm}$$ $$A_{c,eff} = b_w \cdot h_{eff} = 300 \cdot 123.61 = 37082.72\ \mathrm{mm}^2$$ $$\rho_{eff} = \frac{A_s}{A_{c,eff}} = \frac{2454.35}{37082.72} = 0.06619$$ $$e_{sm} - e_{cm} = \frac{\sigma_s - k_t \cdot \frac{f_{ct,eff}}{\rho_{p,eff}} \cdot (1 + a_e \cdot \rho_{p,eff})}{E_s} =$$ $$= \frac{166.07 - 0.4 \cdot \frac{2.56}{0.06619} \cdot (1 + 6.35 \cdot 0.06619)}{200000} = 0.00072$$ $$e_{sm} - e_{cm} \geq 0.6 \cdot \frac{\sigma_s}{E_s} = 0.6 \cdot \frac{166.07}{200000} = 0.00050$$ 4. Calculation of maximum crack spacing $$\text{cover } c = 53.5\ \mathrm{mm}$$ $$k_1 = 0.8$$ $$k_2 = 0.5$$ $$s_{r,max} = 3.4 \cdot c \cdot \frac{0.425 \cdot k_1 \cdot k_2 \cdot diameter}{\rho_{p,eff}} = 3.4 \cdot 53.5 \cdot \frac{0.425 \cdot 0.8 \cdot 0.5 \cdot 25}{0.06619} = 246.11\ \mathrm{mm}$$ $$s_{r,max} \leq 5 \cdot (c + 0.5 \cdot diameter) = 5 \cdot (53.5 + 0.5 \cdot 25) = 330.00\ \mathrm{mm}$$ 5. Calculation of crack width $$w_k = (e_{sm} - e_{cm}) \cdot s_{r,max} = 0.00072 \cdot 246.11 = 0.18\ \mathrm{mm}$$ $$w_k = 0.18\ \mathrm{mm} \leq w_{lim} = 0.3\ \mathrm{mm}$$	**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 $A_c = 210000\ \mathrm{mm}^2$
Tensioning rebar area $A_{sl} = 2454.3500000000004\ \mathrm{mm}^2$
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 $d = 600.40\ \mathrm{mm}$
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

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

$$V_{Rd,max} = \frac{f_{cd} \cdot 0.9 \cdot d \cdot b_w \cdot a_{cw} \cdot v_1}{cot(\theta) + tan(\theta)} = \frac{14.17 \cdot 0.9 \cdot 600.40 \cdot 300 \cdot 1.00 \cdot 0.54}{1.00 + 1.00} = 620.06\ \mathrm{kN}$$

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

$$s_{max} = 0.75 \cdot d = 0.75 \cdot 600.40 = 450.30\ \mathrm{mm}$$

Maximum shear reinforcement

$$A_{sw,max} = \frac{0.5 \cdot a_{cw} \cdot v_1 \cdot f_{cd} \cdot b_w \cdot s}{f_{ywd}} = \frac{0.5 \cdot 1.00 \cdot 0.54 \cdot 14.17 \cdot 300 \cdot 250}{434.78} = 659.81\ \mathrm{mm}^2$$

$$V_{Ed} = 0.00\ \mathrm{kN} \leq V_{Rd,max} = 620.06\ \mathrm{kN}$$

STATUS OK!

$$\rho_w = 0.00209 \geq \rho_{w,min} = 0.00080$$

STATUS OK!

$$A_{sw} = 157.08\ \mathrm{mm}^2 \leq A_{sw,max} = 659.81\ \mathrm{mm}^2$$

STATUS OK!

$$s = 250.00 \text{ mm} \leq s_{max} = 450.30 \text{ mm}$$

STATUS OK!

Deflection check

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

Section input data:
Member span $L = 9500 \text{ mm}$
Load duration factor $\beta = 0.5$
Concrete cement class : R
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 $M = 199.99 \text{ kN-m}$

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

$$I_{uc} = \frac{b \cdot h^3}{12} = \frac{300 \cdot 700^3}{12} = 8575000000.00 \text{ mm}^4$$

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

$$I_{cr} = \frac{b \cdot x^3}{3} + a_e \cdot A_s \cdot (d - x)^2 = \frac{300 \cdot 329.17^3}{3} + 24.43 \cdot 2454.35 \cdot (600.40 - 329.17)^2$$
$$= 7977335134.61 \text{ mm}^4$$

3. Moment that will cause cracking of the section:

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

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

$$M_{cr} = \frac{f_{ctm} \cdot I_{uc}}{y_t} = \frac{2.56 \cdot 8575000000.00}{350.00} = 62.84 \text{ kN-m}$$

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

$$\xi = 1 - \beta \cdot (M_{cr}/M_s^*)^2 = 1 - 0.5 \cdot (62.84/199.99)^2 = 0.95$$

$$(1/r)_{uc} = \frac{M}{E_{c,} \cdot I_{uc}} = \frac{199.994}{8187.21 \cdot 8575000000.00} = 0.00000285 \text{ /mm}$$

$$(1/r)_{cr} = \frac{M}{E_c \cdot I_{cr}} = \frac{199.994}{8187.21 \cdot 7977335134.61} = 0.00000306 \text{ /mm}$$

$$1/r = \xi \cdot (1/r)_{cr} + (1 - \xi) \cdot (1/r)_{uc} = 0.95 \cdot 0.00000306 + (1 - 0.95) \cdot 0.00000285$$
$$= 0.00000305 \text{ /mm}$$

5. Calculate srinkage curvature

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

$$RH_0 = 100 \text{ \%}$$

$$a_{ds1} = 6$$

$$a_{ds2} = 0.11$$

$$\beta_{RH} = 1.55 \cdot \left[1 - (\frac{RH}{RH_0})^3\right] = 1.55 \cdot \left[1 - (\frac{60}{100})^3\right] = 1.2152$$

Basic drying shrinkage strain (B.11)

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

$$= 0.85 \cdot \left[(220 + 110 \cdot 6) \cdot exp(-0.11 \cdot \frac{33.00}{10.00})\right] \cdot 10^{-6} \cdot 1.22 = 0.000632$$

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

$$h_0 = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 210000.00}{2000} = 210.00 \text{ mm}$$

Time-development coefficients (3.10, 3.13)

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0.04 \cdot \sqrt{h_0^3}} = \frac{(450 - 7)}{(450 - 7) + 0.04 \cdot \sqrt{210.00^3}} = 0.78445$$

$$\beta_{as}(t) = 1 - exp(-0.2 \cdot t^{0.5}) = 1 - exp(-0.2 \cdot 450^{0.5}) = 0.985630$$

$$k_h = 0.84$$

$$e_{ca}(t) = \beta_{as}(t) \cdot e_{ca} = 0.9856 \cdot 0.00004 = 0.00004$$

$$e_{cd}(t) = \beta_{ds}(t, t_s) \cdot k_h \cdot e_{cd,0} = 0.78445 \cdot 0.84 \cdot 0.00063 = 0.00042$$

Final shrinkage strain at infinite time

$$e_{cs} = e_{ca}(t) + e_{cd}(t) = 0.00004 + 0.00042 = 0.00045$$

Shrinkage curvature for cracked section

$$S = A_s \cdot (d - x) = 2454.35 \cdot (600.40 - 329.17) = 665686.48 \text{ mm}^3$$

$$(1/r_{cs})_{cr} = e_{cs} \cdot a_e \cdot S/I_{cr} = 0.000454 \cdot 24.43 \cdot 665686.48/7977335134.61 = 0.00000092 \text{ /mm}$$

Shrinkage curvature for uncracked section

$$S = A_s \cdot (d - 0.5 \cdot h) = 2454.35 \cdot (600.40 - 0.5 \cdot 700) = 614569.24 \text{ mm}^3$$

$$(1/r_{cs})_{uc} = e_{cs} \cdot a_e \cdot S/I_{uc} = 0.000454 \cdot 24.43 \cdot 614569.24/8575000000.00 = 0.00000079 \text{ /mm}$$

The average shrinkage curvature

$$(1/r)_{cs} = \xi \cdot (1/r_{cs})_{cr} + (1 - \xi) \cdot (1/r_{cs})_{uc} = 0.95 \cdot 0.00000092 + (1 - 0.95) \cdot 0.00000079 = 0.00000092 \text{ /mm}$$

. Calculate the deflection based on the section curvature

$$\text{Do to loading: } \triangle_1 = k \cdot L^2 \cdot (1/r) = 0.104 \cdot 9500^2 \cdot 0.00000305 = 28.642 \text{ mm}$$

$$\text{Do to shrinkage: } \triangle_2 = k \cdot L^2 \cdot (1/r)_{cs} = 0.104 \cdot 9500^2 \cdot 0.00000092 = 8.618 \text{ mm}$$

$$\text{Total: } \triangle_{total} = \triangle_1 + \triangle_2 = 28.642 + 8.618 = 37.260 \text{ mm}$$

$$\triangle_{total} = 37.26 \text{ mm} \leq (span/250) = (9500/250) = 38.00 \text{ mm}$$

STATUS OK!