REFERENCES	CALCULATIONS	RESULTS

Code: AS 3600-2009

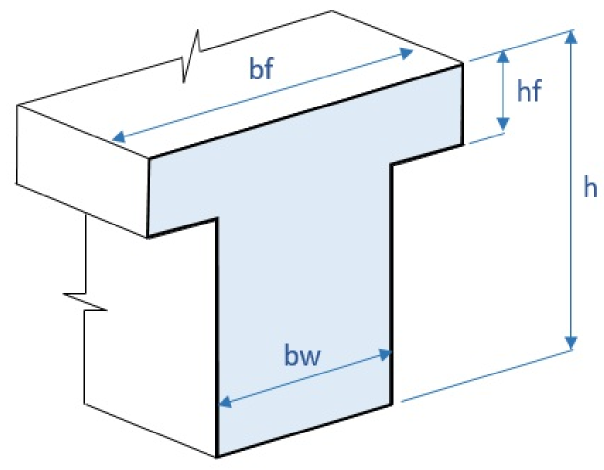
MEMBER #1 (SECTION POSITION 2500.0 mm) BEAM DESIGN REPORT

Project details

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

General member design information

Dimentions:



Height $h = 700 \text{ mm}$
Flange width $b_f = 1100 \text{ mm}$
Flange thickness $h_f = 120 \text{ mm}$
Web width $b_w = 400 \text{ mm}$
Member length = 5000 mm

Material properties:
Concrete strength $\acute{f_c} = 25 \text{ MPa}$
Steel strength of longitudinal rebar $f_{sy} = 500 \text{ MPa}$
Steel strength of shear rebar $f_{syv} = 500 \text{ MPa}$

Design Factors and Settings:
Reinforcement Class : N

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

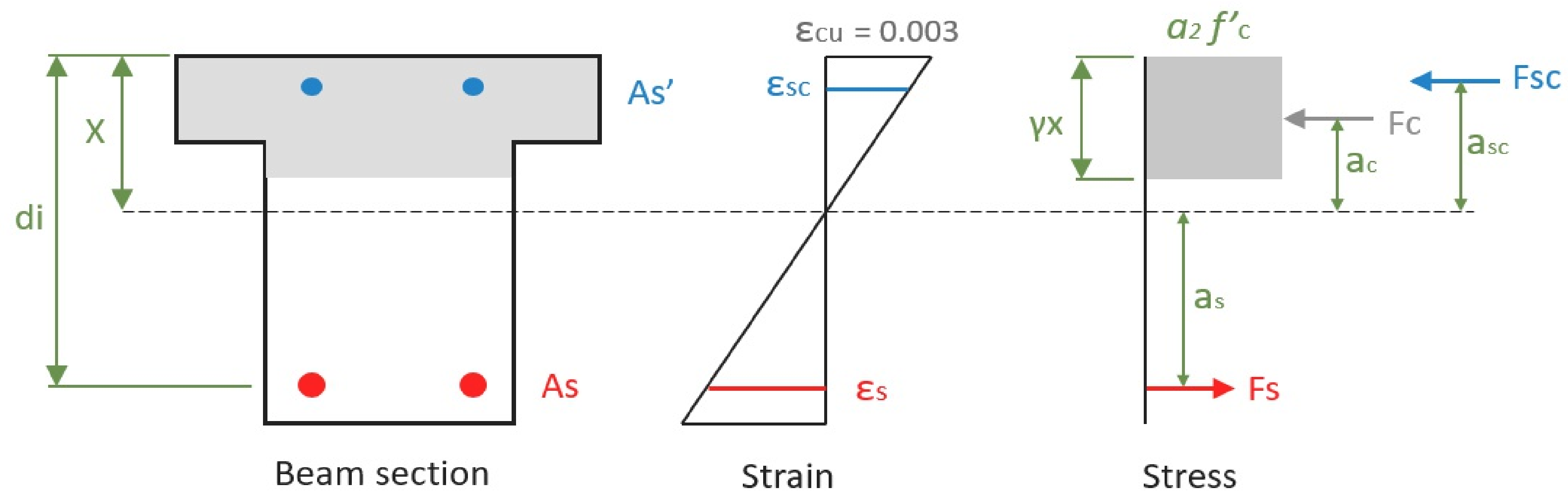
Load Combinations (Serviceability Limit State)

For bending moment in section:
LC1: USER = 150 kN-m

8.1.3(1), 8.1.3(2), Table 2.2.2, 8.1.5, 2.2.2, 8.1.6.1(2)

Flexure check (Positive bending moment case)

BENDING MOMENT CAPACITY



Section input data:
Design yield strain of rebar $e_y = f_{sy}/E_s = 500/200000 = 0.00250$
Ultimate strain in concrete $e_{cu} = 0.003$
Distance to the outermost layer of tensile reinforcement $d_0 = 650 \text{ mm}$
Given bending moment $M^* = 0.00 \text{ kN-m}$

Section Rebar

Depth di (mm)	bar diameter (mm)	bar area Asi (mm2)
650	50.463	2000.03
650	50.463	2000.03
650	50.463	2000.03
650	50.463	2000.03

Rectangular compression block factors (8.1.3(1), 8.1.3(2))

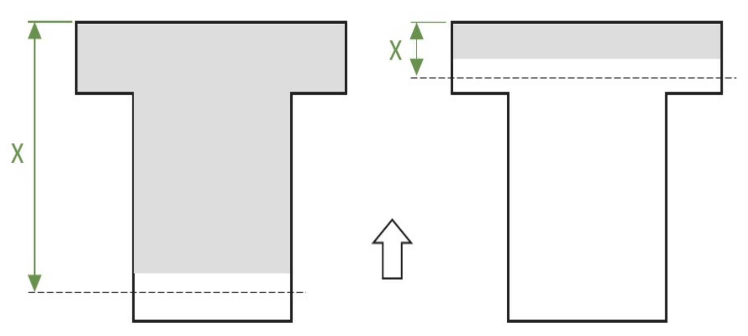
$$\alpha_2 = 1.0 - 0.003 \cdot \acute{f}_c = 1.0 - 0.003 \cdot 25 = 0.93$$

$$\alpha_2 > 0.85 \rightarrow \alpha_2 = 0.85$$

$$\gamma = 1.05 - 0.007 \cdot \acute{f}_c = 1.05 - 0.007 \cdot 25 = 0.88$$

$$\gamma > 0.85 \rightarrow \gamma = 0.85$$

1. Calculation of neutral axis depth x



Calculation is based on iterative process:
- Assume x
- Calculate concrete force $F_c = \alpha_2 \cdot \acute{f}_c \cdot \int_{dA} \cdot \gamma \cdot x$
- Calculate compression force in steel $F_{cs} = \sum \acute{A}_{s,i} \cdot f_{s,i}$
- 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 650 to 0 mm)

Iter.	x (mm)	kuo = x/do	Fc (kN)	Fcs (kN)	Fc + Fcs (kN)	Fs (kN)	Ratio
1	650.0	1.00	6481.25	0.00	6481.25	0.00	Infinity
2	637.0	0.98	6387.32	0.00	6387.32	97.96	65.203
3	624.0	0.96	6293.40	0.00	6293.40	200.00	31.467
4	611.0	0.94	6199.48	0.00	6199.48	306.39	20.234
5	598.0	0.92	6105.55	0.00	6105.55	417.40	14.628
6	585.0	0.90	6011.63	0.00	6011.63	533.34	11.272
7	572.0	0.88	5917.70	0.00	5917.70	654.56	9.041
8	559.0	0.86	5823.77	0.00	5823.77	781.41	7.453
9	546.0	0.84	5729.85	0.00	5729.85	914.30	6.267
10	533.0	0.82	5635.93	0.00	5635.93	1053.67	5.349
11	520.0	0.80	5542.00	0.00	5542.00	1200.02	4.618
12	507.0	0.78	5448.07	0.00	5448.07	1353.87	4.024
13	494.0	0.76	5354.15	0.00	5354.15	1515.81	3.532
14	481.0	0.74	5260.23	0.00	5260.23	1686.51	3.119
15	468.0	0.72	5166.30	0.00	5166.30	1866.69	2.768
16	455.0	0.70	5072.38	0.00	5072.38	2057.17	2.466
17	442.0	0.68	4978.45	0.00	4978.45	2258.86	2.204

18	429.0	0.66	4884.52	0.00	4884.52	2472.76	1.975
19	416.0	0.64	4790.60	0.00	4790.60	2700.04	1.774
20	403.0	0.62	4696.68	0.00	4696.68	2941.98	1.596
21	390.0	0.60	4602.75	0.00	4602.75	3200.05	1.438
22	377.0	0.58	4508.82	0.00	4508.82	3475.91	1.297
23	364.0	0.56	4414.90	0.00	4414.90	3771.49	1.171
24	351.0	0.54	4320.98	0.00	4320.98	4000.06	1.080
25	338.0	0.52	4227.05	0.00	4227.05	4000.06	1.057
26	325.0	0.50	4133.13	0.00	4133.13	4000.06	1.033
27	312.0	0.48	4039.20	0.00	4039.20	4000.06	1.010
(Fc + Fcs) < Fs. Updating of iterations							
1	299.0	0.46	3945.28	0.00	3945.28	4000.06	0.986
2	311.7	0.48	4037.32	0.00	4037.32	4000.06	1.009
3	311.5	0.48	4035.44	0.00	4035.44	4000.06	1.009
4	311.2	0.48	4033.56	0.00	4033.56	4000.06	1.008
5	311.0	0.48	4031.69	0.00	4031.69	4000.06	1.008
6	310.7	0.48	4029.81	0.00	4029.81	4000.06	1.007
7	310.4	0.48	4027.93	0.00	4027.93	4000.06	1.007
8	310.2	0.48	4026.05	0.00	4026.05	4000.06	1.006
9	309.9	0.48	4024.17	0.00	4024.17	4000.06	1.006
10	309.7	0.48	4022.29	0.00	4022.29	4000.06	1.006
11	309.4	0.48	4020.42	0.00	4020.42	4000.06	1.005
12	309.1	0.48	4018.54	0.00	4018.54	4000.06	1.005
13	308.9	0.48	4016.66	0.00	4016.66	4000.06	1.004
14	308.6	0.47	4014.78	0.00	4014.78	4000.06	1.004
15	308.4	0.47	4012.90	0.00	4012.90	4000.06	1.003
16	308.1	0.47	4011.02	0.00	4011.02	4000.06	1.003
17	307.8	0.47	4009.14	0.00	4009.14	4000.06	1.002
18	307.6	0.47	4007.27	0.00	4007.27	4000.06	1.002
19	307.3	0.47	4005.39	0.00	4005.39	4000.06	1.001
20	307.1	0.47	4003.51	0.00	4003.51	4000.06	1.001
21	306.8	0.47	4001.63	0.00	4001.63	4000.06	1.000
22	306.5	0.47	3999.75	0.00	3999.75	4000.06	1.000

Final value of x is 306.54 mm and flexural tension reinforcement area is 8000.12 mm2
Working depth of reinforcement $d = 650.00\ \mathrm{mm}$

Strength reduction factor for reinforcement Class N in bending without axial tension or compression (Table 2.2.2)

$$\phi = 1.19 - 13 \cdot k_{uo}/12 = 1.19 - 13 \cdot 0.47/12 = 0.68$$

Check maximum allowable depth of the rectangular compression block (8.1.5)

$$a = \gamma \cdot x = 0.85 \cdot 306.54 = 260.56\ \mathrm{mm} > a_{max} = \gamma \cdot k_u \cdot d_0 = 0.85 \cdot 0.36 \cdot 650 = 198.90\ \mathrm{mm}$$

Check if the compression reinforcement was applied

2. Calculation moment resistance M_d (2.2.2)

$$\phi M_d = (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) \cdot \phi = (830.45 + 0.00 + 1373.86) \cdot 0.68 = 1496.95\ \mathrm{kN\text{-}m}$$

$$M^* = 0.00\ \mathrm{kN\text{-}m} \leq M_d = 1496.95\ \mathrm{kN\text{-}m}\ \mathrm{(Ratio:\ 0.000)}$$

STATUS OK!

Ratio: 0.000

3. Minimum required flexural tension reinforcement in a beam section (8.1.6.1(2))

$$f'_{ct,f} = 0.6\sqrt{f'_c} = 0.6\sqrt{25} = 3.00$$

for T-Sections with the web in tension

$$\alpha_b = 0.2 + (\frac{b_f}{b_w} - 1) \cdot (0.4 \cdot (\frac{h_f}{h}) - 0.18) = 0.2 + (\frac{1100}{400} - 1) \cdot (0.4 \cdot (\frac{120}{700}) - 0.18) = 0.01$$

$$\alpha_b < 0.2 \cdot (\frac{b_f}{b_w})^{1/4} = 0.2 \cdot (\frac{1100}{400})^{1/4} = 0.26$$

$$\alpha_b = 0.26$$

$$A_{st,min} = \alpha_b \cdot (\frac{h}{d} \cdot \frac{f'_{ct,f}}{f_{sy}}) \cdot b_w \cdot d = 0.26 \cdot (\frac{700}{650.00}) \cdot \frac{3.00}{500} \cdot 400 \cdot 650.00 = 432.69 \text{ mm}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$A_{st,max} = 0.04 \cdot b_w \cdot d = 0.04 \cdot 400 \cdot 650.00 = 10400.00 \text{ mm}^2$$

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

$$A_{st} = 8000.12 \text{ mm}^2 \leq A_{st,max} = 10400.00 \text{ mm}^2 \text{ (Ratio: 0.769)}$$

STATUS OK!
Ratio: 0.769

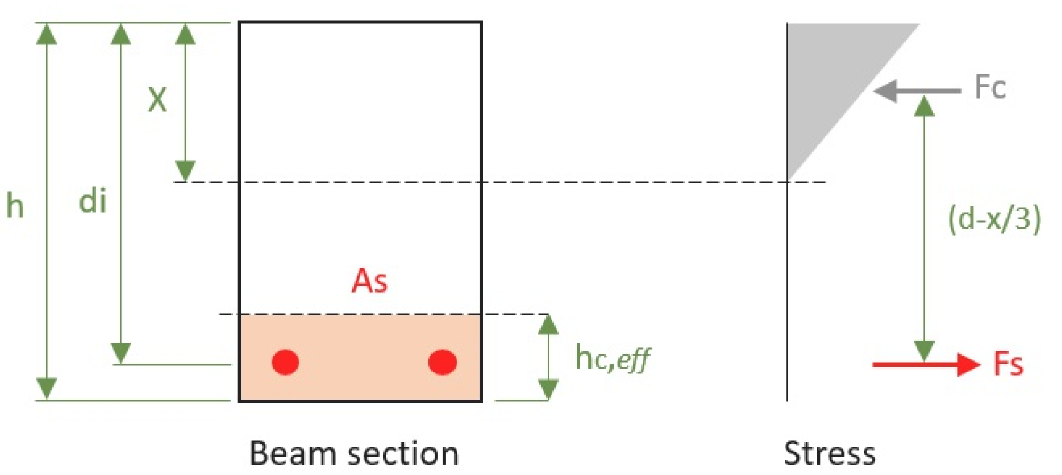
$$A_{st} = 8000.12 \text{ mm}^2 \geq A_{st,min} = 432.69 \text{ mm}^2 \text{ (Ratio: 0.054)}$$

STATUS OK!
Ratio: 0.054

Crack control check (Positive bending moment case)

3.1.8.3, 8.6.1

CRACK CONTROL OF BEAMS



Section input data:
Section concrete area $A_g = 364000 \text{ mm}^2$
Web width $b_w = 400 \text{ mm}$
Age of concrete at loading $\tau = 3 \text{ days}$
Age of concrete at the moment considered $t = 10000 \text{ days}$
Environment type factor $k_4 = 0.7$
Modulus of elasticity of concrete $E_c = 26700.00 \text{ MPa}$
Given bending moment $M_s^* = 150.00 \text{ kN-m}$

1. Design creep coefficient ϕ_{cc} (3.1.8.3)

$$\text{Basic creep coefficient } \phi_{cc,b} = 4.20$$

Notional size of the member

$$t_h = \frac{2 \cdot A_c}{u} = \frac{2 \cdot 364000.00}{3600} = 202.22 \text{ mm}$$

$$\alpha_2 = 1.0 + 1.12 \cdot e^{-0.008 \cdot t_h} = 1.0 + 1.12 \cdot e^{-0.008 \cdot 202.22} = 1.22$$

$$k_2 = \frac{\alpha_2 \cdot t^{0.8}}{t^{0.8} + 0.15 \cdot t_h} = \frac{1.22 \cdot 10000^{0.8}}{10000^{0.8} + 0.15 \cdot 202.22} = 1.20$$

$$k_3 = 2.7/\left[1.0 + log(\tau)\right] = 2.7/\left[1.0 + log(3)\right] = 1.29$$

$$\acute{f_c} \leq 50 \rightarrow k_5 = 1.0$$

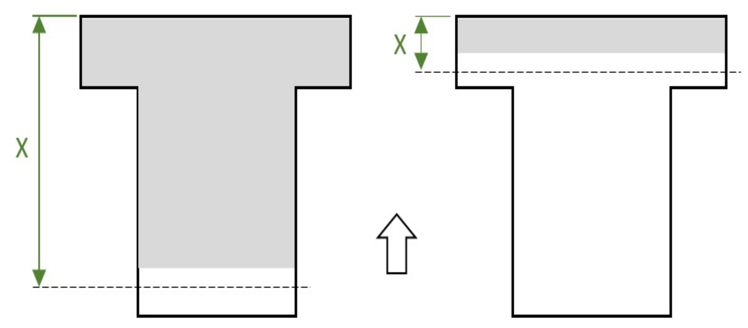
$$\phi_{cc} = k_2 \cdot k_3 \cdot k_4 \cdot k_5 \cdot \phi_{cc,b} = 1.20 \cdot 1.29 \cdot 0.70 \cdot 1.00 \cdot 4.20 = 4.54$$

Effective modulus

$$E_{eff} = \frac{E_{cm}}{1 + \phi_{cc}} = \frac{26700.00}{1 + 4.54} = 4823.04 \text{ MPa}$$

$$a_e = \frac{E_s}{E_{eff}} = \frac{200000}{4823.04} = 41.47$$

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 650 to 0 mm)

Iter.	x (mm)	As (mm2)	Left force equil. part (kN)	Right force equil. part (kN)	Ratio
1	650.00	0.00	305174.94	439234.94	1.439
2	637.00	8000.12	301828.74	427137.84	1.415
3	624.00	8000.12	298550.14	415175.94	1.391
4	611.00	8000.12	295339.14	403349.24	1.366
5	598.00	8000.12	292195.74	391657.74	1.340
6	585.00	8000.12	289119.94	380101.44	1.315
7	572.00	8000.12	286111.74	368680.34	1.289
8	559.00	8000.12	283171.14	357394.45	1.262
9	546.00	8000.12	280298.14	346243.75	1.235
10	533.00	8000.12	277492.74	335228.25	1.208
11	520.00	8000.12	274754.94	324347.95	1.180
12	507.00	8000.12	272084.74	313602.85	1.153
13	494.00	8000.12	269482.14	302992.95	1.124
14	481.00	8000.12	266947.14	292518.25	1.096
15	468.00	8000.12	264479.74	282178.75	1.067
16	455.00	8000.12	262079.94	271974.46	1.038
17	442.00	8000.12	259747.74	261905.36	1.008
left part < right part. Updating of iterations					
1	429.00	8000.12	257483.14	251971.46	0.979
2	441.74	8000.12	259701.78	261705.35	1.008
3	441.48	8000.12	259655.85	261505.41	1.007
4	441.22	8000.12	259609.95	261305.51	1.007
5	440.96	8000.12	259564.08	261105.67	1.006
6	440.70	8000.12	259518.23	260905.88	1.005
7	440.44	8000.12	259472.42	260706.15	1.005
8	440.18	8000.12	259426.62	260506.47	1.004
9	439.92	8000.12	259380.86	260306.85	1.004
10	439.66	8000.12	259335.12	260107.28	1.003
11	439.40	8000.12	259289.41	259907.76	1.002
12	439.14	8000.12	259243.72	259708.30	1.002
13	438.88	8000.12	259198.07	259508.89	1.001
14	438.62	8000.12	259152.44	259309.54	1.001
15	438.36	8000.12	259106.83	259110.24	1.000
16	438.10	8000.12	259061.26	258910.99	0.999

Final value of x is 438.10 mm and tensioning rebar area is $8000.12\ \mathrm{mm}^2$
Working depth of reinforcement $d = 650.00\ \mathrm{mm}$

3. Calculation of stress in tensionng zone of reinforcement

$$\sigma_{scr} = \frac{M^*}{A_s \cdot (d - x/3)} = \frac{150000000}{8000.12 \cdot (650.00 - 438.10/3)} = 37.20\ \mathrm{MPa}$$

4. Limiting stress check (8.6.1)

$$\text{For nominal bar diameter in tension zone } d_b \\ = 50.463\ \mathrm{mm} \text{ the maximum stress is } 120.00\ \mathrm{MPa}\ (\text{Table } 8.6.1(\mathrm{A}))$$

For centre-to-centre spacing in tension zone 75.00 mm the maximum stress is 340.00 MPa (Table 8.6.1(B))

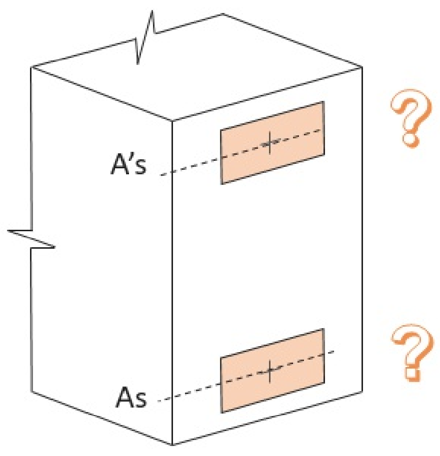
$$\sigma_{scr} = 37.20 \text{ MPa} \leq 120.00 \text{ MPa (For nominal bar diameter) (Ratio: 0.310)}$$

STATUS OK!
Ratio: 0.310

$$\sigma_{scr} = 37.20 \text{ MPa} \leq 340.00 \text{ MPa (For centre-to-centre spacing) (Ratio: 0.109)}$$

STATUS OK!
Ratio: 0.109

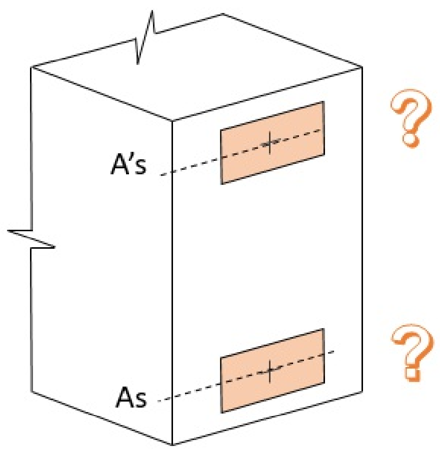
Flexure check (Negative bending moment case)



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 control 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!

8.2.7.1, 8.2.6, 8.2.9

Shear check

SHEAR FORCE CAPACITY (Members without shear reinforcement)

Section input data:
Mean width of web $b_w = 400 \text{ mm}$
Section concrete area $A_g = 364000 \text{ mm}^2$
Tensioning rebar area $A_{st} = 8000.12 \text{ mm}^2$
Working depth of reinforcement $d_0 = 650 \text{ mm}$
Given shear force $V^* = 0.00 \text{ kN}$
Corresponding axial force $N^* = 0.00 \text{ kN}$

1. Determine concrete shear capacity (8.2.7.1)

$$f'_{cv} = (f'_c)^{1/3} = (25)^{1/3} = 2.92 \text{ MPa} \leq 4 \text{ MPa}$$

$$\beta_1 = 1.1 \cdot (1.6 - \frac{d_0}{1000}) = 1.1 \cdot (1.6 - \frac{650}{1000}) = 1.05 < 1.1 \rightarrow \beta_1 = 1.1$$

$$\beta_2 = 1.0$$

$$\beta_3 = 1.0$$

	$$V_{uc} = \beta_1 \cdot \beta_2 \cdot \beta_3 \cdot b_w \cdot d_0 \cdot f'_{cv} \cdot (\frac{A_{st}}{b_w \cdot d_0})^{1/3} = 1.10 \cdot 1.00 \cdot 1.00 \cdot 400 \cdot 650 \cdot 2.92 \cdot (\frac{8000.12}{400 \cdot 650})^{1/3}$$ $$= 262.05 \text{ kN}$$ $$V^* = 0 \text{ kN} \leq V_{cv} = 262.05 \text{ kN}$$	**STATUS OK!**