REFERENCES	CALCULATIONS	RESULTS

Code: AS 3600:2018(A1)

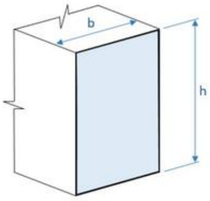
MEMBER #2 (SECTION POSITION 1250.0 mm) BEAM DESIGN REPORT

Project details

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

General member design information

Dimentions:



Height $h = 199.9\ \mathrm{mm}$
Width $b = 254\ \mathrm{mm}$
Member length $= 2500\ \mathrm{mm}$

Material properties:
Concrete strength $f_c' = 25\ \mathrm{MPa}$
Steel strength of longitudinal rebar $f_{sy} = 500\ \mathrm{MPa}$
Steel strength of shear rebar $f_{syv} = 500\ \mathrm{MPa}$
Limit crack width $w_{max}' = 0.3\ \mathrm{mm}$

Design Factors and Settings:
Reinforcement Class : N

Load Combinations

Ultimate Limit State:
LC 1: ULS: 1. D (M = 0.34 kN-m, V = 0.00 kN)
LC 2: ULS: 2. D + L (M = 0.24 kN-m, V = 0.93 kN)
LC 3: ULS: 3. D + (S or Lr or R) (M = 0.34 kN-m, V = 0.00 kN)
LC 4: ULS: 4. D + 0.75L + 0.75(S or Lr or R) (M = 0.27 kN-m, V = 0.70 kN)
LC 5: ULS: 5a. D + 0.6W (M = 0.34 kN-m, V = 0.00 kN)
LC 6: ULS: 5b. D + 0.7E (M = 0.34 kN-m, V = 0.00 kN)
LC 7: ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R) (M = 0.27 kN-m, V = 0.70 kN)
LC 8: ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S (M = 0.27 kN-m, V = 0.70 kN)
LC 9: ULS: 7. 0.6D + 0.6W (M = 0.20 kN-m, V = 0.00 kN)
LC 10: ULS: 8. 0.6D + 0.7E (M = 0.20 kN-m, V = 0.00 kN)

Serviceabiity Limit State:
LC 1: LC-1 (M = 0.34 kN-m)

Accepted forces for section check:
Positive moment strength case : (M = 0.34 kN-m)
Positive moment service. case: (M = 0.34 kN-m)
Negative moment strength case: (M = 0.00 kN-m)
Negative moment service. case: (M = 0.00 kN-m)
Shear strength case: M = (0.24 kN-m, V = 0.93 kN)

DL - Dead Load
LL - Live Load
WL - Wind Load
LrL - Roof Live Load
RL - Rain Load
SL - Snow Load
EL - Earthquake Load

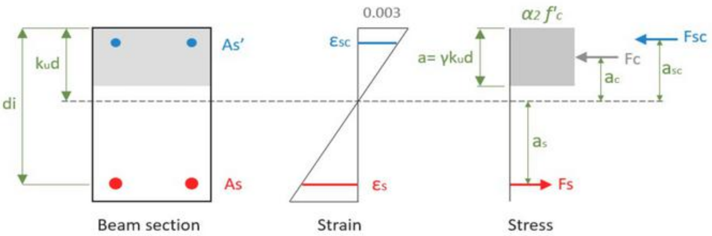
Flexure check (Positive bending moment case)

Intermediate results of the optimization

Iteration	Ratio	Status
1	0.289	OK

Results of iteration 1

BENDING MOMENT CAPACITY

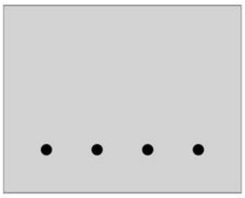


Section input data:
Design yield strain of rebar $e_y = f_{sy}/E_s = 500/200000 = 0.00250$
Ultimate strain in concrete $e_{cu} = 0.003$
Distance to the outermost layer of tensile reinforcement $d_0 = 153.9$ mm
Given bending moment $M^* = 0.34$ kN-m

Section Rebar

Depth d_i (mm)	bar diameter (mm)	bar area A_{si} (mm^2)
153.9	12	113.10
153.9	12	113.10
153.9	12	113.10
153.9	12	113.10

Reinforced section view



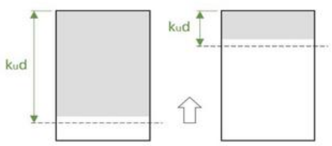
8.1.3(1), 8.1.3(2)

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

$$\alpha_2 = 0.85 - 0.0015 \cdot f'_c = 0.85 - 0.0015 \cdot 25 = 0.81$$

$$\gamma = 0.97 - 0.0025 \cdot f'_c = 0.97 - 0.0025 \cdot 25 = 0.91$$

1. Calculation of neutral axis depth $k_u d$



Calculation is based on iterative process:
- Assume kud
- Calculate concrete force $F_c = \alpha_2 \cdot f'_c \cdot \int_{dA} \cdot \gamma \cdot k_u \cdot d$
- 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 $e_s = e_{cu} \cdot (k_u \cdot d - d)/k_u \cdot d$
Reinforcement strains below axis $e_s = e_{cu} \cdot (d - k_u \cdot d)/k_u \cdot d$

Searching of neutral axis kud (from 153.9 to 0 mm)

Iter.	$k_u d$ (mm)	$k_{uo} = a/d_o$	F_c (kN)	F_{cs} (kN)	$F_c + F_{cs}$ (kN)	F_s (kN)	Ratio
17	48.3	0.31	225.97	0.00	225.97	226.19	0.999

Final value of $k_u d$ is 48.26 mm and flexural tension reinforcement area is 452.39 mm^2
Working depth of reinforcement d = 153.90 mm

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

$$\phi = 1.24 - 13 \cdot k_{uo}/12 = 1.24 - 13 \cdot 0.31/12 = 0.90$$

$$\phi > 0.85 \rightarrow \phi = 0.85$$

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

$$a = \gamma \cdot k_u d = 0.91 \cdot 48.26 = 43.80 \text{ mm} \leq a_{max} = \gamma \cdot k_u \cdot d_0 = 0.91 \cdot 0.36 \cdot 153.9 = 50.28 \text{ mm}$$

2. Calculation moment resistance

$$\phi M_u = (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) \cdot \phi = (5.96 + 0.00 + 23.89) \cdot 0.85 = 25.37 \text{ kN-m}$$

$$M^* = 0.34 \text{ kN-m} \leq \phi M_u = 25.37 \text{ kN-m (Ratio: 0.013)}$$

STATUS OK!
Ratio: 0.013

8.1.6.1(1)

3. Minimum required strength in bending $(M_{uo})_{min}$ (8.1.6.1(1))

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

$$I_g = \frac{b \cdot h^3}{12} = \frac{254 \cdot 199.9^3}{12} = 169079460.31 \text{ mm}^4$$

$$Z = \frac{I_g}{y} = \frac{169079460.31}{99.95} = 1691640.42 \text{ mm}^3$$

$$M_{uo,min} = 1.2 \cdot Z \cdot f'_{ct,f} = 1.2 \cdot (1691640.42 \cdot 3.00) \cdot 10^{-6} = 6.09 \text{ kN-m}$$

$$M_{uo,min} = 6.09 \text{ kN-m} \leq \phi M_u = 25.37 \text{ kN-m (Ratio: 0.240)}$$

STATUS OK!
Ratio: 0.240

8.1.6.1(2)

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

$$\alpha_b = 0.2$$

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

$$A_{st,min} = \left[\alpha_b \cdot (h/d)^2 \cdot (f'_{ct,f}/f_{sy})\right] \cdot b_w \cdot d = \left[0.20 \cdot (199.90/153.90)^2 \cdot (3.00/500.00)\right] \cdot 254.00 \cdot 153.90 = 79.14 \text{ mm}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$A_{st,max} = 0.04 \cdot b \cdot d = 0.04 \cdot 254 \cdot 153.90 = 1563.62 \text{ mm}^2$$

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

$$A_{st} = 452.39 \text{ mm}^2 \leq A_{st,max} = 1563.62 \text{ mm}^2 \text{ (Ratio: 0.289)}$$

$$A_{st} = 452.39 \text{ mm}^2 \geq A_{st,min} = 79.14 \text{ mm}^2 \text{ (Ratio: 0.175)}$$

STATUS OK!
Ratio: 0.289

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