REFERENCES	CALCULATIONS	RESULTS

MEMBER #1 (SECTION POSITION 120.0 INCHES) BEAM DESIGN REPORT

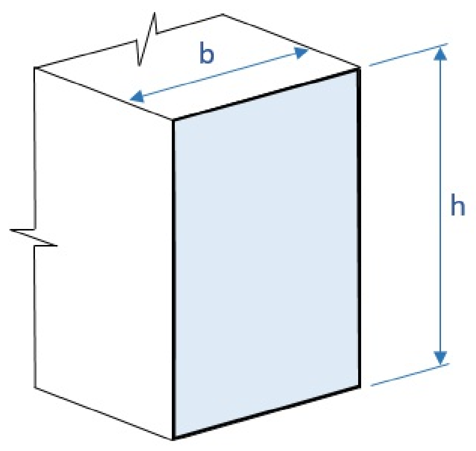
Code: ACI 318-14

Project details

Project Name: null
Project ID: null
Company: null
Designer: null
Client: null
Project Notes: null
Project Units: Imperial

General member design information

Dimentions:



Height $h = 20 \text{ in}$
Width $b = 12 \text{ in}$
Member length = 240 in

Material properties:
Concrete strength $f_c = 4000 \text{ psi}$
Steel strength of longitudinal rebar $f_y = 60000 \text{ psi}$
Steel strength of shear rebar $f_{yt} = 50000 \text{ psi}$
Permissible crack width $c_w = 0.012 \text{ in}$

Load Combinations

Ultimate Limit State:
LC 1: 1.4DL (M = 63.00 kip-ft, V = 0.00 Kip)
LC 2: 1.2DL+1.6LL (M = 214.00 kip-ft, V = 9.60 Kip)

Serviceabiity Limit State:
LC 1: 1.0DL (M = 45.00 Kip-ft)
LC 2: 1.0DL+1.0LL (M = 145.00 Kip-ft)

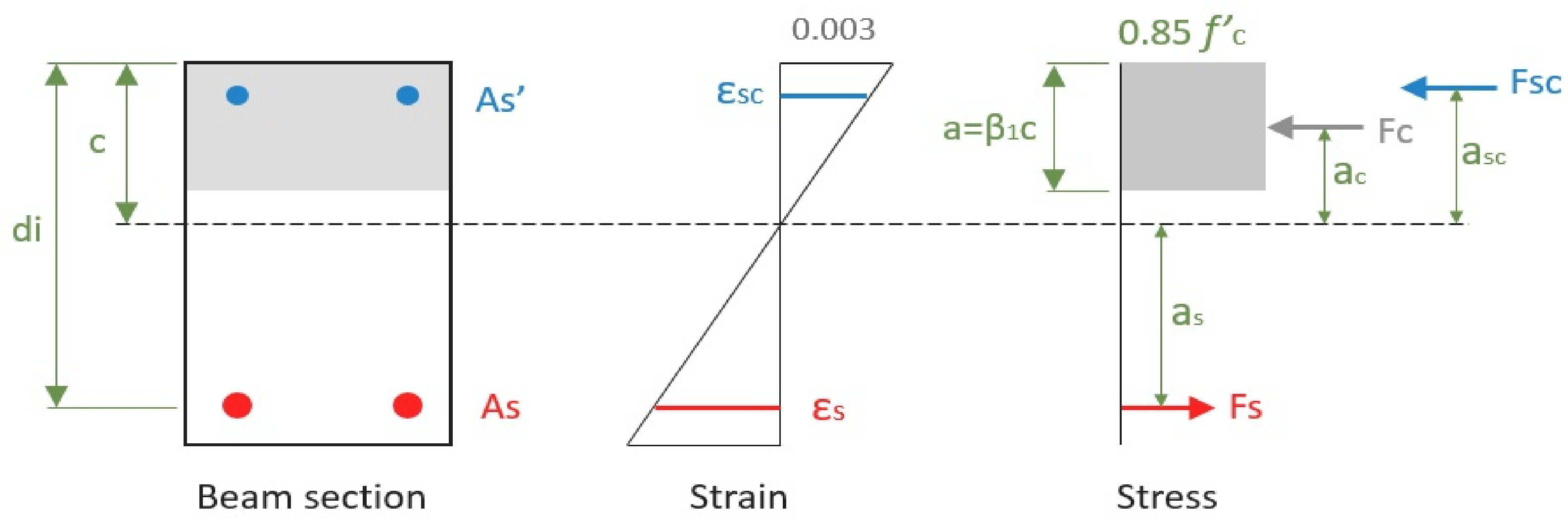
Accepted forces for section check:
Positive moment strength case : (M = 214.00 Kip-ft)
Positive moment service. case: (M = 145.00 Kip-ft)
Negative moment strength case: (M = 0.00 Kip-ft)
Negative moment service. case: (M = 0.00 Kip-ft)
Shear strength case: M = (214.00 Kip-ft, V = 9.60 Kip)

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)

CHAPTER 9 (Section 9.5)

BENDING MOMENT CAPACITY



Section input data:
Design yield strain of rebar $e_y = f_y / E_s = 60000/29000000 = 0.00207$
Ultimate strain in concrete $e_c = 0.003$
Distance to the outermost layer of tensile reinforcement $d_t = 17.5 \text{ in}$
Given bending moment $M = 214.00 \text{ kip-ft}$

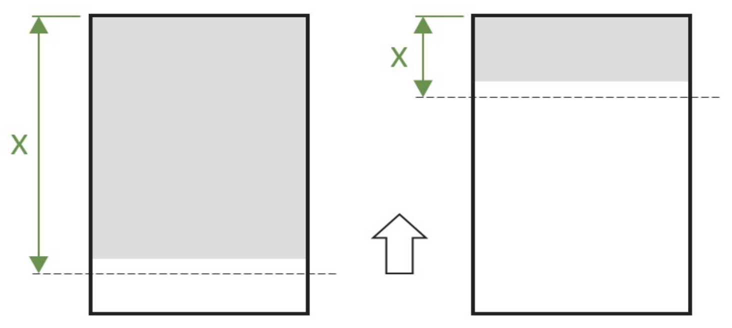
Section Rebar

Depth di (in)	bar diameter (in)	bar area Asi (in^2)
17.5	1	0.79
17.5	1	0.79
17.5	1	0.79
17.5	1	0.79

Rectangular compression block factor

$$4000 \text{ psi} \le f_c \le 8000 \text{ psi} \rightarrow \beta_1 = 0.85 - (\frac{f_c - 4000}{1000}) \cdot 0.05 = 0.85 - (\frac{4000 - 4000}{1000}) \cdot 0.05 = 0.85$$

1. Calculation of neutral axis depth c



Calculation is based on iterative process:
- Assume c
- Calculate concrete force $F_c = 0.85 \cdot f_c' \cdot \int_{dA} \cdot \beta_1 \cdot c$
- 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 $e_s = e_c \cdot (c - d)/c$
Reinforcement strains below axis $e_s = e_c \cdot (d - c)/c$

Searching of neutral axis c (from 17.5 to 0 in)

Iter.	c (in)	c/dt	Fc (lbf)	Fcs (lbf)	Fc + Fcs (lbf)	Fs (lbf)	Ratio
1	17.5	1.00	606900.00	0.00	606900.00	0.00	Infinity
2	17.1	0.98	594762.00	0.00	594762.00	5610.61	106.007
3	16.8	0.96	582624.00	0.00	582624.00	11455.00	50.862
4	16.4	0.94	570486.00	0.00	570486.00	17548.09	32.510
5	16.1	0.92	558348.00	0.00	558348.00	23906.09	23.356
6	15.7	0.90	546210.00	0.00	546210.00	30546.67	17.881
7	15.4	0.88	534072.00	0.00	534072.00	37489.09	14.246
8	15.0	0.86	521934.00	0.00	521934.00	44754.42	11.662
9	14.7	0.84	509796.00	0.00	509796.00	52365.71	9.735
10	14.3	0.82	497658.00	0.00	497658.00	60348.29	8.246
11	14.0	0.80	485520.00	0.00	485520.00	68730.00	7.064
12	13.6	0.78	473382.00	0.00	473382.00	77541.54	6.105
13	13.3	0.76	461244.00	0.00	461244.00	86816.84	5.313
14	12.9	0.74	449106.00	0.00	449106.00	96593.51	4.649
15	12.6	0.72	436968.00	0.00	436968.00	106913.33	4.087
16	12.2	0.70	424830.00	0.00	424830.00	117822.86	3.606
17	11.9	0.68	412692.00	0.00	412692.00	129374.12	3.190
18	11.5	0.66	400554.00	0.00	400554.00	141625.45	2.828
19	11.2	0.64	388416.00	0.00	388416.00	154642.50	2.512
20	10.8	0.62	376278.00	0.00	376278.00	168499.35	2.233
21	10.5	0.60	364140.00	0.00	364140.00	183280.00	1.987
22	10.2	0.58	352002.00	0.00	352002.00	189600.00	1.857

23	9.8	0.56	339864.00	0.00	339864.00	189600.00	1.793
24	9.5	0.54	327726.00	0.00	327726.00	189600.00	1.729
25	9.1	0.52	315588.00	0.00	315588.00	189600.00	1.664
26	8.8	0.50	303450.00	0.00	303450.00	189600.00	1.600
27	8.4	0.48	291312.00	0.00	291312.00	189600.00	1.536
28	8.1	0.46	279174.00	0.00	279174.00	189600.00	1.472
29	7.7	0.44	267036.00	0.00	267036.00	189600.00	1.408
30	7.4	0.42	254898.00	0.00	254898.00	189600.00	1.344
31	7.0	0.40	242760.00	0.00	242760.00	189600.00	1.280
32	6.7	0.38	230622.00	0.00	230622.00	189600.00	1.216
33	6.3	0.36	218484.00	0.00	218484.00	189600.00	1.152
34	6.0	0.34	206346.00	0.00	206346.00	189600.00	1.088
35	5.6	0.32	194208.00	0.00	194208.00	189600.00	1.024
(Fc + Fcs) < Fs. Updating of iterations							
1	5.3	0.30	182070.00	0.00	182070.00	189600.00	0.960
2	5.6	0.32	193965.24	0.00	193965.24	189600.00	1.023
3	5.6	0.32	193722.48	0.00	193722.48	189600.00	1.022
4	5.6	0.32	193479.72	0.00	193479.72	189600.00	1.020
5	5.6	0.32	193236.96	0.00	193236.96	189600.00	1.019
6	5.6	0.32	192994.20	0.00	192994.20	189600.00	1.018
7	5.6	0.32	192751.44	0.00	192751.44	189600.00	1.017
8	5.6	0.32	192508.68	0.00	192508.68	189600.00	1.015
9	5.5	0.32	192265.92	0.00	192265.92	189600.00	1.014
10	5.5	0.32	192023.16	0.00	192023.16	189600.00	1.013
11	5.5	0.32	191780.40	0.00	191780.40	189600.00	1.012
12	5.5	0.32	191537.64	0.00	191537.64	189600.00	1.010
13	5.5	0.32	191294.88	0.00	191294.88	189600.00	1.009
14	5.5	0.31	191052.12	0.00	191052.12	189600.00	1.008
15	5.5	0.31	190809.36	0.00	190809.36	189600.00	1.006
16	5.5	0.31	190566.60	0.00	190566.60	189600.00	1.005
17	5.5	0.31	190323.84	0.00	190323.84	189600.00	1.004
18	5.5	0.31	190081.08	0.00	190081.08	189600.00	1.003
19	5.5	0.31	189838.32	0.00	189838.32	189600.00	1.001
20	5.47	0.31	189595.56	0.00	189595.56	189600.00	1.000

Final value of c is 5.47 in and flexural tension reinforcement area is 3.16 in^2
Working depth of reinforcement $d = 17.50 \text{ in}$

$$e_t = \frac{d_t - c}{c} \cdot (0.003) = \frac{17.5 - 5.47}{5.47} \cdot (0.003) = 0.00660 \geq 0.005 \rightarrow \phi = 0.9$$

2. Calculation moment resistance M_R

$$M_R = \phi \cdot M = \phi \cdot (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) = 0.90 \cdot (595998.38 + 0.00 + 2281456.80) = 2589.71 \text{ Ibf-in} \\ = 215.81 \text{ Kip-ft}$$

$$M = 214.00 \text{ Kip-ft} \leq M_R = 215.81 \text{ Kip-ft}$$

STATUS OK!

3. Minimum required flexural tension reinforcement in a beam section

$$A_{s,min} = \frac{3 \cdot \sqrt{f_c}}{f_y} \cdot b_w \cdot d = \frac{3 \cdot \sqrt{4000}}{60000} \cdot 12 \cdot 17.50 = 0.66\ \text{in}^2$$

$$A_{s,min} \leq \frac{200}{f_y} \cdot b_w \cdot d = \frac{200}{60000} \cdot 12 \cdot 17.50 = 0.70\ \text{in}^2 \rightarrow A_{s,min} = 0.70\ \text{in}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$\rho_b = 0.85 \cdot \beta_1 \cdot \frac{f_c}{f_y} \cdot (\frac{87}{87 + f_y}) = 0.85 \cdot 0.85 \cdot \frac{4000}{60000} \cdot (\frac{87}{87 + 60000}) = 0.02851$$

$$\rho_{max} = (\frac{0.003 + (f_y / E_s)}{0.008}) \cdot \rho_b = (\frac{0.003 + (60000/29000000)}{0.008}) \cdot 0.02851 = 0.01806$$

$$A_{s,max} = \rho_{max} \cdot b_w \cdot d = 0.01806 \cdot 12 \cdot 17.50 = 3.79\ \text{in}^2$$

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

$$A_{st} = 3.16\ \text{mm}^2 \leq A_{st,max} = 3.79\ \text{mm}^2$$

STATUS OK!

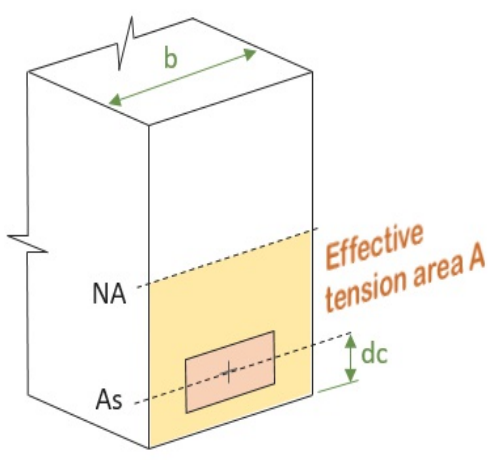
$$A_{st} = 3.16\ \text{mm}^2 \geq A_{st,min} = 0.70\ \text{mm}^2$$

STATUS OK!

CHAPTER 10 (Section 10.6)

Crack check (Positive bending moment case)

CONTROL OF FLEXURAL AND MISCELLANEOUS CRACKS



Section input data:
Permissible crack widths w_{lim} = 0.012 in
Ratio of the distance β_h = 1.2
Cover of the outermost bar d_c = 2.5 in
Effective tension area of concrete around the main reinforcing A = 15.00 in^2
Clear cover c_c = 2 in

1. Determine permitted steel stress, f_s

$$f_s = 0.6 \cdot f_y = 36000.00\ \text{psi}$$

2. Determine estimated cracking width, w

$$w = 0.076 \cdot \beta_h \cdot f_s \cdot \sqrt[3]{d_c \cdot A} = 0.076 \cdot 1.2 \cdot 36000.00 \cdot \sqrt[3]{2.5 \cdot 15.00} = 0.0110\ \text{in}$$

3. Determine maximum code-permitted bar spacing, s

$$s = 15 \cdot (\frac{40000}{f_s}) - 2.5 \cdot c_c = 15 \cdot (\frac{40000}{36000.00}) - 2.5 \cdot 2 = 11.67\ \text{in}$$

$$s \leq 12 \cdot (\frac{40000}{f_s}) = 12 \cdot (\frac{40000}{36000.00}) = 13.33\ \text{in}$$

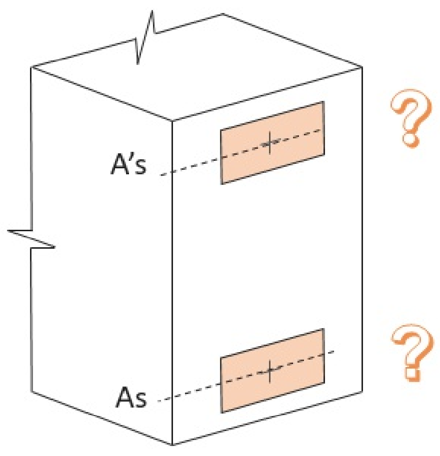
$$w = 0.0110\ \text{in} \leq w_{lim} = 0.012\ \text{in}$$

STATUS OK!

$$c_c = 2\ \text{in} \leq s = 11.67\ \text{in}$$

STATUS OK!

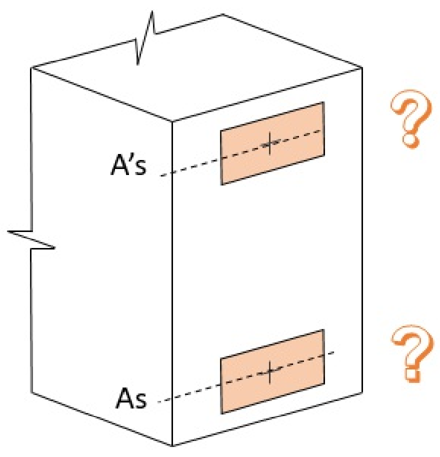
Flexure check (Negative bending moment case)



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 check (Negative bending moment case)



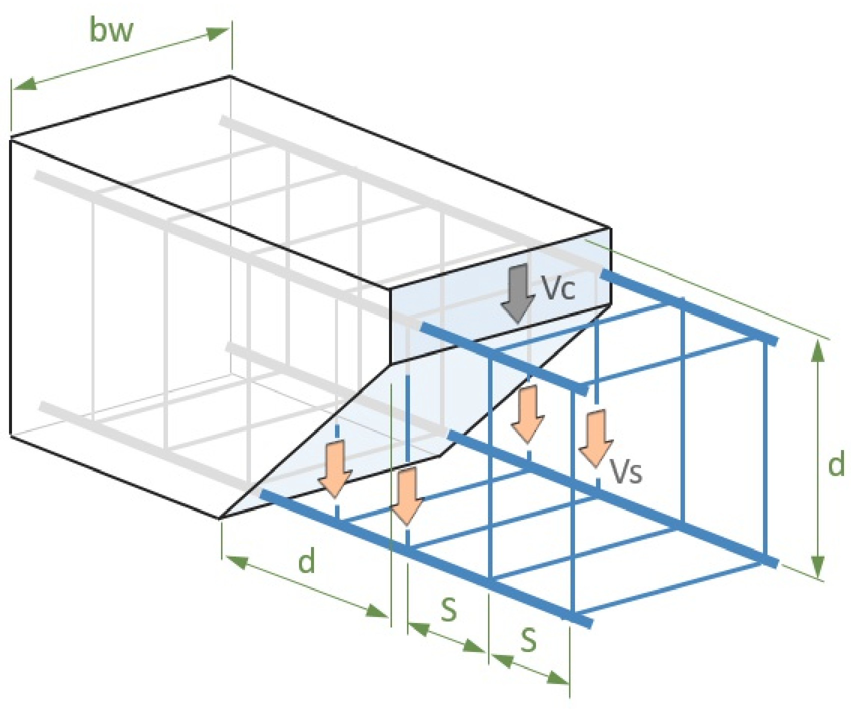
Bottom Reinforcement is absent in the section. Design checks can't be performed. But as acting moment value is equal to zero no need to check.

STATUS OK!

CHAPTER 11 (Section 11.2)

Shear check

BEAM ANALYSIS FOR SHEAR (IN PLANE)



Section input data:
Working depth of reinforcement d = 17.5 in
Spacing of the stirrups s = 10 in
Number of stirrups that are intersected by inclined section n = 1
Web width b_w = 12 in
Tensile reinforcement area A_s = 3.16 in^2
Sum of the cross sectional areas of the stirrup legs A_v = 0.39 in^2
Concrete area A_g = 240.00 in^2
Given axial force according to Load Combination (-) N_u = 0.00 kip
Given bending moment according to Load Combination (1.2DL+1.6LL) M_u = 214.00 kip-ft
Given shear force according to Load Combination (1.2DL+1.6LL) V_u = 9.60 kip

1. Calculate minimum area of shear reinforcement ($A_{v,min}$)

$$A_{v,min} = 0.75 \cdot \sqrt{f_c} \cdot \frac{b_w \cdot s}{f_y} = 0.75 \cdot \sqrt{4000} \cdot \frac{12 \cdot 10}{50000} = 0.11\,\text{in}^2$$

$$A_{v,min} = 0.11\,\text{in}^2 < \frac{50 \cdot b_w \cdot s}{f_y} = \frac{50 \cdot 12 \cdot 10}{50000} = 0.12\,\text{in}^2 \rightarrow A_{v,min} = 0.12\,\text{in}^2$$

$$A_v = 0.39\,\text{in}^2 \geq A_{v,min} = 0.12\,\text{in}^2 \rightarrow \text{ area of shear reinforcement is satisfied}$$

STATUS OK!

2. Calculate maximum spacing for vertical stirrups (s_{max})

$$S_{max} = \min\left[\frac{d}{2}, 24\right] = 8.75 \text{ in}$$

$$s = 10 \text{ in} > S_{max} = 8.75 \text{ in} \rightarrow \text{ spacing of stirrups is not satisfied}$$

STATUS NG!

3. Calculate shear strength of section stirrups (V_s)

$$V_s = n \cdot A_v \cdot f_y = 1 \cdot 0.39 \cdot 50000 = 19634.95 \text{ lb} = 19.63 \text{ kip}$$

4. Calculate shear strength of concrete section (V_c)
case beam subjected to shear force only

$$V_c = 2 \cdot \lambda \cdot \sqrt{f_c} \cdot b_w \cdot d =$$

$$= 2 \cdot 1 \cdot \sqrt{4000} \cdot 12 \cdot 17.5 = 26563.13 \text{ lb} = 26.56 \text{ kip}$$

case beam subjected to flexure and shear

$$\lambda = 1.0$$

$$\rho_w = \frac{A_s}{b_w \cdot d} = \frac{3.16}{12 \cdot 17.5} = 0.02$$

$$\frac{V_u \cdot d}{M_u} = \frac{9.60 \cdot 17.5}{214.00} = 0.07 \leq 1.0$$

$$V_c = (1.9 \cdot \lambda \cdot \sqrt{f_c} + 2.5 \cdot \rho_w \cdot \frac{V_u \cdot d}{M_u}) \cdot b_w \cdot d =$$

$$= (1.9 \cdot 1 \cdot \sqrt{4000} + 2.5 \cdot 0.02 \cdot 0.07) \cdot 12 \cdot 17.5 = 25235.49 \text{ lb} = 25.24 \text{ kip}$$

$$V_c = 25235.49 \text{ lb} = 25.24 \text{ kip} \leq 3.5 \cdot \lambda \cdot \sqrt{f_c} \cdot b_w \cdot d =$$

$$= 3.5 \cdot 1 \cdot \sqrt{4000} \cdot 12 \cdot 17.5 = 46485.48 \text{ lb} = 46.49 \text{ kip}$$

$$V_c = 25235.49 \text{ lb} = 25.24 \text{ kip}$$

For design check V_c is taken as 25.24 kip

5. Calculate design resisting shear (V_R)

$$V_R = \phi \cdot (V_c + V_s) = 0.75 \cdot (25235.49 + 19634.95) = 33.65 \text{ kip}$$

$$V_u = 9.60 \text{ kip} \leq V_R = 33.65 \text{ kip}$$

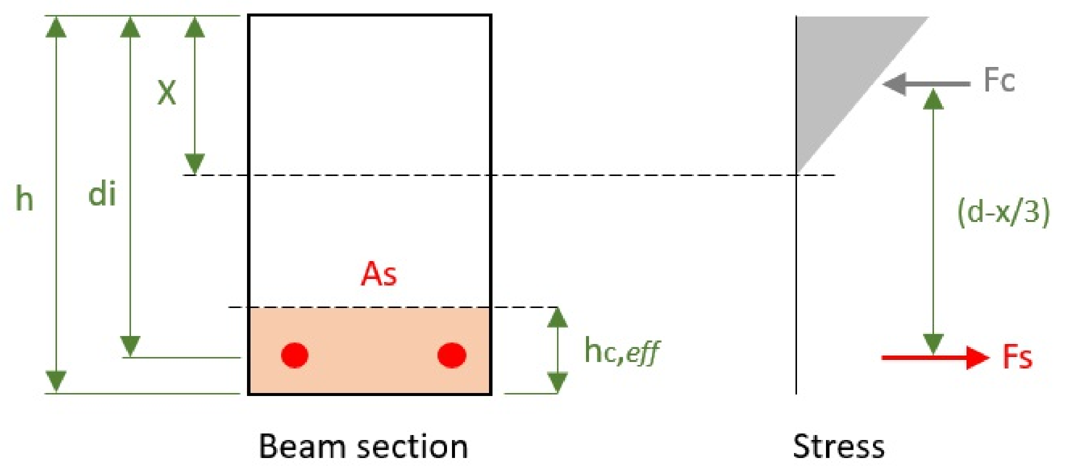
STATUS OK!

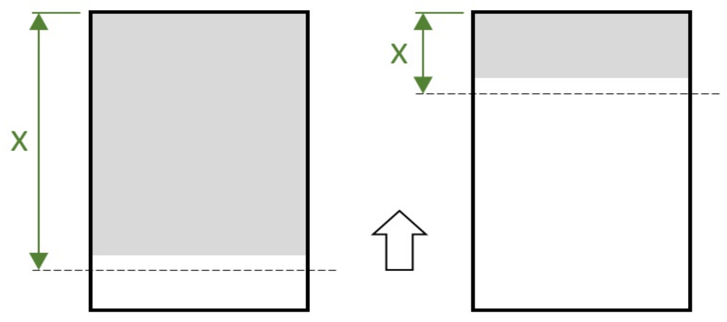
Deflection check

DEFLECTION OF BEAM (immediate and long-time)

Section input data:
Weight concrete factor $\lambda = 1$
Given service bending moment due to Dead Load $M_{a,DL} = 45.00$ Kip-ft
Given service bending moment due to Live Load $M_{a,LL} = 100.00$ Kip-ft
Typical dflection factor $k = 0.104$
Long-term factor $\xi = 2$
Live Load deflection limitation $L/180$
Long term deflection limitation $L/240$
Sustained Live Load: $50.0\ \%$
Steel Modulus of Elasticity $E_s = 29000000$ psi
Concrete Modulus of Elasticity $E_c = 57000 \cdot \sqrt{f'_c} = \sqrt{4000} = 3604996.53$ psi
Member span $L = 240$ in
Modular ratio $n = E_s/E_c = 8.04$

1. 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 $A_{comp.} + a_e \cdot A_s + a_e \cdot \acute{A_s}$
- Check Ratio: (left part / right part) = 1.0

Searching of neutral axis x (from 17.5 to 0 in)

Iter.	x (in)	As (in2)	Asc (in2)	Left force equil. part (lb)	Right force equil. part (lb)	Ratio
1	17.50	0.00	0.00	2282.35	4119.85	1.805
2	17.15	3.16	0.00	2209.59	3965.43	1.795
3	16.80	3.16	0.00	2138.29	3813.94	1.784
4	16.45	3.16	0.00	2068.47	3665.39	1.772
5	16.10	3.16	0.00	2000.11	3519.79	1.760
6	15.75	3.16	0.00	1933.23	3377.12	1.747
7	15.40	3.16	0.00	1867.81	3237.39	1.733
8	15.05	3.16	0.00	1803.87	3100.61	1.719
9	14.70	3.16	0.00	1741.39	2966.76	1.704
10	14.35	3.16	0.00	1680.39	2835.85	1.688
11	14.00	3.16	0.00	1620.85	2707.88	1.671
12	13.65	3.16	0.00	1562.79	2582.86	1.653
13	13.30	3.16	0.00	1506.19	2460.77	1.634
14	12.95	3.16	0.00	1451.07	2341.62	1.614
15	12.60	3.16	0.00	1397.41	2225.42	1.593
16	12.25	3.16	0.00	1345.23	2112.15	1.570
17	11.90	3.16	0.00	1294.51	2001.82	1.546
18	11.55	3.16	0.00	1245.27	1894.43	1.521
19	11.20	3.16	0.00	1197.49	1789.99	1.495
20	10.85	3.16	0.00	1151.19	1688.48	1.467
21	10.50	3.16	0.00	1106.35	1589.91	1.437
22	10.15	3.16	0.00	1062.99	1494.29	1.406
23	9.80	3.16	0.00	1021.09	1401.60	1.373
24	9.45	3.16	0.00	980.67	1311.85	1.338
25	9.10	3.16	0.00	941.71	1225.04	1.301
26	8.75	3.16	0.00	904.23	1141.18	1.262
27	8.40	3.16	0.00	868.21	1060.25	1.221
28	8.05	3.16	0.00	833.67	982.26	1.178

29	7.70	3.16	0.00	800.59	907.22	1.133
30	7.35	3.16	0.00	768.99	835.11	1.086
31	7.00	3.16	0.00	738.85	765.94	1.037
left part < right part. Updating of iterations						
1	6.65	3.16	0.00	710.19	699.71	0.985
2	6.99	3.16	0.00	738.27	764.59	1.036
3	6.99	3.16	0.00	737.68	763.24	1.035
4	6.98	3.16	0.00	737.09	761.89	1.034
5	6.97	3.16	0.00	736.51	760.54	1.033
6	6.97	3.16	0.00	735.92	759.19	1.032
7	6.96	3.16	0.00	735.34	757.84	1.031
8	6.95	3.16	0.00	734.75	756.49	1.030
9	6.94	3.16	0.00	734.17	755.15	1.029
10	6.94	3.16	0.00	733.59	753.80	1.028
11	6.93	3.16	0.00	733.00	752.46	1.027
12	6.92	3.16	0.00	732.42	751.12	1.026
13	6.92	3.16	0.00	731.84	749.78	1.025
14	6.91	3.16	0.00	731.26	748.44	1.023
15	6.90	3.16	0.00	730.68	747.10	1.022
16	6.90	3.16	0.00	730.10	745.77	1.021
17	6.89	3.16	0.00	729.52	744.43	1.020
18	6.88	3.16	0.00	728.94	743.09	1.019
19	6.87	3.16	0.00	728.37	741.76	1.018
20	6.87	3.16	0.00	727.79	740.43	1.017
21	6.86	3.16	0.00	727.21	739.10	1.016
22	6.85	3.16	0.00	726.64	737.77	1.015
23	6.85	3.16	0.00	726.06	736.44	1.014
24	6.84	3.16	0.00	725.49	735.11	1.013
25	6.83	3.16	0.00	724.91	733.79	1.012
26	6.83	3.16	0.00	724.34	732.46	1.011
27	6.82	3.16	0.00	723.77	731.14	1.010
28	6.81	3.16	0.00	723.19	729.81	1.009
29	6.80	3.16	0.00	722.62	728.49	1.008
30	6.80	3.16	0.00	722.05	727.17	1.007
31	6.79	3.16	0.00	721.48	725.85	1.006
32	6.78	3.16	0.00	720.91	724.53	1.005
33	6.78	3.16	0.00	720.34	723.22	1.004
34	6.77	3.16	0.00	719.77	721.90	1.003
35	6.76	3.16	0.00	719.20	720.59	1.002
36	6.76	3.16	0.00	718.63	719.27	1.001
37	6.75	3.16	0.00	718.07	717.96	1.00

Value of x is $6.75\ \mathrm{in}$
Tensioning rebar area $A_s = 3.16\ \mathrm{in}^2$
Compression rebar area $A_{sc} = 0.00\ \mathrm{in}^2$
Working depth of reinforcement $d = 17.50\ \mathrm{in}$

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

$$I_g = \frac{b \cdot h^3}{12} = \frac{12 \cdot 20^3}{12} = 8000.00 \text{ in}^4$$

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

$$I_{cr} = \frac{b \cdot x^3}{3} + a_e \cdot A_s \cdot (d - x)^2 = \frac{12 \cdot 6.75^3}{3} + 8.04 \cdot 3.16 \cdot (17.50 - 6.75)^2 = 4167.82 \text{ in}^4$$

4. Moment that will cause cracking of the section:

$$f_r = 7.5 \cdot \lambda \cdot \sqrt{\acute{f_c}} = 7.5 \cdot 1.00 \cdot \sqrt{4000} = 474.34 \text{ psi}$$

$$M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{474.34 \cdot 0.001 \cdot 8000.00}{10.00 \cdot 12} = 31.62 \text{ Kip-ft}$$

5. Calculate the effective moment of inertia

$$I_e = \left(\frac{M_{cr}}{(M_{a,DL} + M_{a,LL})}\right)^3 \cdot I_g + \left[1 - \left(\frac{M_{cr}}{(M_{a,DL} + M_{a,LL})}\right)^3\right] \cdot I_{cr} = \left(\frac{31.62}{145.00}\right)^3 \cdot 8000.00 + \left[1 - \left(\frac{31.62}{145.00}\right)^3\right] \cdot 4167.82$$
$$= 4207.57 \text{ in}^4$$

$$I_e \le I_g$$

6. Calculate the curvature of section

$$\text{Immediate Dead Load: } (1/r)_{e,DL} = \frac{M_a^*}{E_c \cdot I_e} = \frac{45.00 \cdot 12}{3604996.53 \cdot 0.001 \cdot 4207.57} = 0.0000356006 \text{ /in}$$

$$\text{Immediate Live Load: } (1/r)_{e,LL} = \frac{M_a^*}{E_c \cdot I_e} = \frac{100.00 \cdot 12}{3604996.53 \cdot 0.001 \cdot 4207.57} = 0.0000791125 \text{ /in}$$

7. Calculate the deflection based on section curvature

$$\text{Immediate Dead Load: } \triangle_{DL} = k \cdot L^2 \cdot (1/r)_{e,DL} = 0.104 \cdot 240^2 \cdot 0.00003560 = 0.2133 \text{ in}$$

$$\text{Immediate Live Load: } \triangle_{LL} = k \cdot L^2 \cdot (1/r)_{e,LL} = 0.104 \cdot 240^2 \cdot 0.00007911 = 0.4739 \text{ in}$$

$$\text{Immediate total: } \triangle = \triangle_{DL} + \triangle_{LL} = 0.2133 + 0.4739 = 0.69 \text{ in}$$

8. Calculate the additional long-time deflection

$$\acute{\rho} = \frac{\acute{A_s}}{b \cdot d} = \frac{0.000}{12 \cdot 17.50} = 0.00000 \text{ in}$$

$$\lambda_{\triangle} = \frac{\xi}{(1 + 50 \cdot \acute{\rho})} = \frac{2.00}{(1 + 50 \cdot 0.00000)} = 2.00$$

$$\text{Long-time: } \triangle_{LT} = \lambda_{\triangle} \cdot (0.5 \cdot \lambda_{\triangle_{LL}} + \triangle_{DL}) + \triangle_{LL} = 2.00 \cdot (0.5 \cdot 0.47 + 0.21) + 0.47 = 1.3744 \text{ in}$$

9. Check with limited deflection

$$\triangle_{LL} = 0.4739 \text{ in} \le \frac{L}{180} = \frac{240.00}{180} = 1.33 \text{ in}$$

STATUS OK!

$$\triangle_{LT} = 1.3744 \text{ in} > \frac{L}{240} = \frac{240.00}{240} = 1.00 \text{ in}$$

STATUS NG!