REFERENCES	CALCULATIONS	RESULTS

MEMBER #1 (SECTION POSITION 180.0 INCHES) BEAM DESIGN REPORT

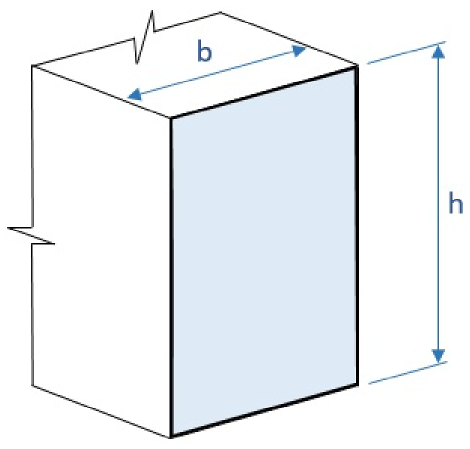
Code: ACI 318-14

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: Imperial

General member design information

Dimentions:



Height $h = 16.5 \text{ in}$
Width $b = 10 \text{ in}$
Member length = 360 in

Material properties:
Concrete strength $f_c = 3000 \text{ psi}$
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 = 27.99 kip-ft, V = 0.00 Kip)
LC 2: 1.2DL+1.6LL (M = 47.99 kip-ft, V = 0.00 Kip)
LC 3: 0.9DL+1.6WL (M = 17.99 kip-ft, V = 0.00 Kip)
LC 4: 1.2DL+1.0LL+1.6WL (M = 38.99 kip-ft, V = 0.00 Kip)
LC 5: 0.9DL+1.0EL (M = 17.99 kip-ft, V = 0.00 Kip)
LC 6: 1.2DL+1.0LL+1.0EL (M = 38.99 kip-ft, V = 0.00 Kip)
LC 7: 1.2DL+1.6LL+0.5SL (M = 47.99 kip-ft, V = 0.00 Kip)
LC 8: 1.2DL+1.0LL+1.6SL (M = 38.99 kip-ft, V = 0.00 Kip)
LC 9: 1.2DL+1.6SL+0.8WL (M = 23.99 kip-ft, V = 0.00 Kip)
LC 10: 1.2DL+1.0LL+0.5SL+1.6WL (M = 38.99 kip-ft, V = 0.00 Kip)
LC 11: 1.2DL+1.0LL+0.2SL+1.0EL (M = 38.99 kip-ft, V = 0.00 Kip)

Serviceabiity Limit State:
LC 1: 1.0DL (M = 19.99 Kip-ft)
LC 2: 1.0DL+1.0LL (M = 34.99 Kip-ft)

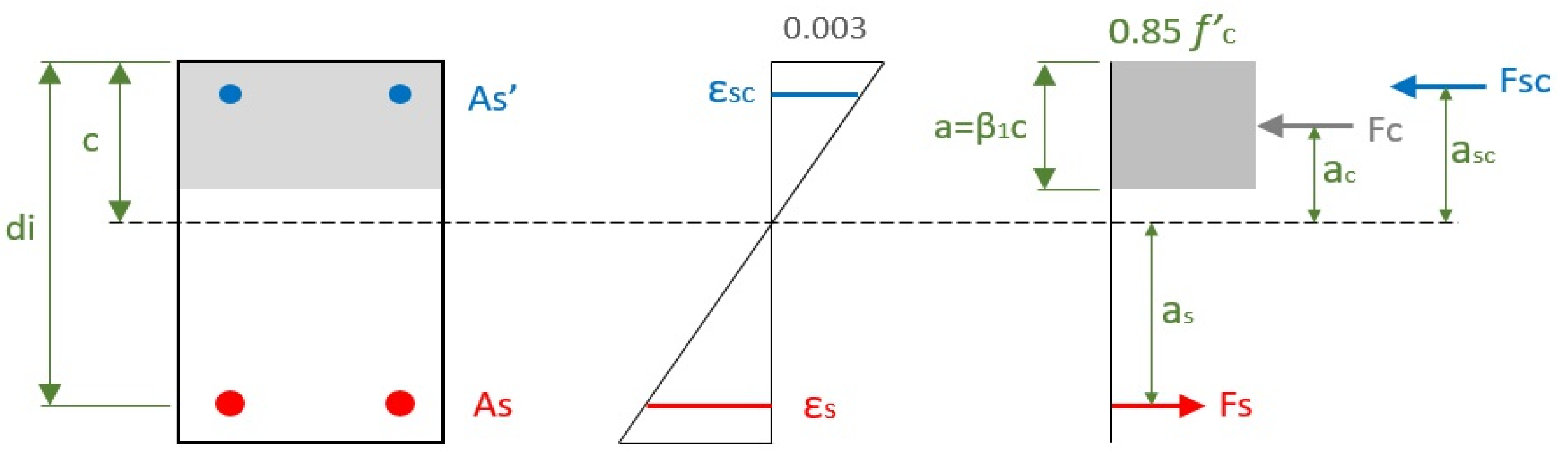
Accepted forces for section check:
Positive moment strength case : (M = 47.99 Kip-ft)
Positive moment service. case: (M = 34.99 Kip-ft)
Negative moment strength case: (M = 0.00 Kip-ft)
Negative moment service. case: (M = 0.00 Kip-ft)
Shear strength case: M = (47.99 Kip-ft, V = 0.00 Kip)

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



Beam section Strain Stress

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 $d_t = 14 \text{ in}$
Given bending moment $M = 47.99 \text{ kip-ft}$

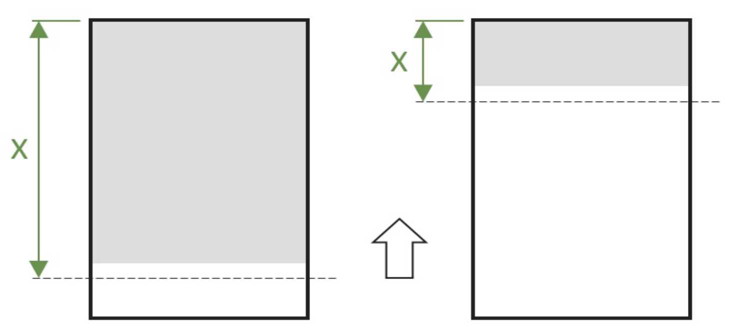
Section Rebar

Depth di (in)	bar diameter (in)	bar area Asi (in^2)
14.00	1.000	0.79
14.00	1.000	0.79
14.00	1.000	0.79

Rectangular compression block factor

$$f_c < 4000 \text{ psi} \rightarrow \beta_1 = 0.85$$

1. Calculation of neutral axis depth c



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

Searching of neutral axis c (from 14 to 0 in)

Iter.	c (in)	c/dt	Fc (lbf)	Fcs (lbf)	Fc + Fcs (lbf)	Fs (lbf)	Ratio
1	14.0	1.00	303450.00	0.00	303450.00	0.00	Infinity
2	13.7	0.98	297381.00	0.00	297381.00	4207.96	70.671
3	13.4	0.96	291312.00	0.00	291312.00	8591.25	33.908
4	13.2	0.94	285243.00	0.00	285243.00	13161.06	21.673
5	12.9	0.92	279174.00	0.00	279174.00	17929.57	15.571
6	12.6	0.90	273105.00	0.00	273105.00	22910.00	11.921
7	12.3	0.88	267036.00	0.00	267036.00	28116.82	9.497
8	12.0	0.86	260967.00	0.00	260967.00	33565.81	7.775
9	11.8	0.84	254898.00	0.00	254898.00	39274.29	6.490
10	11.5	0.82	248829.00	0.00	248829.00	45261.22	5.498
11	11.2	0.80	242760.00	0.00	242760.00	51547.50	4.709
12	10.9	0.78	236691.00	0.00	236691.00	58156.15	4.070
13	10.6	0.76	230622.00	0.00	230622.00	65112.63	3.542
14	10.4	0.74	224553.00	0.00	224553.00	72445.14	3.100
15	10.1	0.72	218484.00	0.00	218484.00	80185.00	2.725

16	9.8	0.70	212415.00	0.00	212415.00	88367.14	2.404
17	9.5	0.68	206346.00	0.00	206346.00	97030.59	2.127
18	9.2	0.66	200277.00	0.00	200277.00	106219.09	1.886
19	9.0	0.64	194208.00	0.00	194208.00	115981.87	1.674
20	8.7	0.62	188139.00	0.00	188139.00	126374.52	1.489
21	8.4	0.60	182070.00	0.00	182070.00	137460.00	1.325
22	8.1	0.58	176001.00	0.00	176001.00	142200.00	1.238
23	7.8	0.56	169932.00	0.00	169932.00	142200.00	1.195
24	7.6	0.54	163863.00	0.00	163863.00	142200.00	1.152
25	7.3	0.52	157794.00	0.00	157794.00	142200.00	1.110
26	7.0	0.50	151725.00	0.00	151725.00	142200.00	1.067
27	6.7	0.48	145656.00	0.00	145656.00	142200.00	1.024
(Fc + Fcs) < Fs. Updating of iterations							
1	6.4	0.46	139587.00	0.00	139587.00	142200.00	0.982
2	6.7	0.48	145534.62	0.00	145534.62	142200.00	1.023
3	6.7	0.48	145413.24	0.00	145413.24	142200.00	1.023
4	6.7	0.48	145291.86	0.00	145291.86	142200.00	1.022
5	6.7	0.48	145170.48	0.00	145170.48	142200.00	1.021
6	6.7	0.48	145049.10	0.00	145049.10	142200.00	1.020
7	6.7	0.48	144927.72	0.00	144927.72	142200.00	1.019
8	6.7	0.48	144806.34	0.00	144806.34	142200.00	1.018
9	6.7	0.48	144684.96	0.00	144684.96	142200.00	1.017
10	6.7	0.48	144563.58	0.00	144563.58	142200.00	1.017
11	6.7	0.48	144442.20	0.00	144442.20	142200.00	1.016
12	6.7	0.48	144320.82	0.00	144320.82	142200.00	1.015
13	6.7	0.48	144199.44	0.00	144199.44	142200.00	1.014
14	6.6	0.47	144078.06	0.00	144078.06	142200.00	1.013
15	6.6	0.47	143956.68	0.00	143956.68	142200.00	1.012
16	6.6	0.47	143835.30	0.00	143835.30	142200.00	1.012
17	6.6	0.47	143713.92	0.00	143713.92	142200.00	1.011
18	6.6	0.47	143592.54	0.00	143592.54	142200.00	1.010
19	6.6	0.47	143471.16	0.00	143471.16	142200.00	1.009
20	6.6	0.47	143349.78	0.00	143349.78	142200.00	1.008
21	6.6	0.47	143228.40	0.00	143228.40	142200.00	1.007
22	6.6	0.47	143107.02	0.00	143107.02	142200.00	1.006
23	6.6	0.47	142985.64	0.00	142985.64	142200.00	1.006
24	6.6	0.47	142864.26	0.00	142864.26	142200.00	1.005
25	6.6	0.47	142742.88	0.00	142742.88	142200.00	1.004
26	6.6	0.47	142621.50	0.00	142621.50	142200.00	1.003
27	6.6	0.47	142500.12	0.00	142500.12	142200.00	1.002
28	6.6	0.47	142378.74	0.00	142378.74	142200.00	1.001
29	6.6	0.47	142257.36	0.00	142257.36	142200.00	1.000
30	6.56	0.47	142135.98	0.00	142135.98	142200.00	1.000

Final value of c is 6.56 in and flexural tension reinforcement area is 2.37 in^2

Working depth of reinforcement $d = 14.00 \text{ in}$

$$e_t = \frac{d_t - c}{c} \cdot (0.003) = \frac{14 - 6.56}{6.56} \cdot (0.003) = 0.00340$$

$$e_y < e_t < 0.005$$

$$\phi = 0.65 + (\frac{0.9 - 0.65}{0.005 - e_y}) \cdot (e_t - e_y) = 0.65 + (\frac{0.9 - 0.65}{0.005 - 0.00207}) \cdot (0.00340 - 0.00207) = 0.76$$

2. Calculation moment resistance M_R

$$M_R = \phi \cdot M = \phi \cdot (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) = 0.76 \cdot (535940.77 + 0.00 + 1058309.28) = 1217.91 \text{ Ibf-in} = 101.49 \text{ Kip-ft}$$

$$M = 47.99 \text{ Kip-ft} \leq M_R = 101.49 \text{ Kip-ft}$$

STATUS OK!

3. Minimum required flexural tension reinforcement in a beam section

$$A_{s,min} = \frac{3 \cdot \sqrt{f_c}}{f_y} \cdot b_w \cdot d = \frac{3 \cdot \sqrt{3000}}{60000} \cdot 10 \cdot 14.00 = 0.38 \text{ in}^2$$

$$A_{s,min} \leq \frac{200}{f_y} \cdot b_w \cdot d = \frac{200}{60000} \cdot 10 \cdot 14.00 = 0.47 \text{ in}^2 \rightarrow A_{s,min} = 0.47 \text{ in}^2$$

4. Maximum required flexural tension reinforcement in a beam section

$$\rho_b = 0.85 \cdot \beta_1 \cdot \frac{f_c}{f_y} \cdot (\frac{87}{87 + f_y}) = 0.85 \cdot 0.85 \cdot \frac{3000}{60000} \cdot (\frac{87}{87 + 60000}) = 0.02138$$

$$\rho_{max} = (\frac{0.003 + (f_y/E_s)}{0.008}) \cdot \rho_b = (\frac{0.003 + (60000/29000000)}{0.008}) \cdot 0.02138 = 0.01355$$

$$A_{s,max} = \rho_{max} \cdot b_w \cdot d = 0.01355 \cdot 10 \cdot 14.00 = 1.90 \text{ in}^2$$

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

$$A_{st} = 2.37 \text{ mm}^2 > A_{st,max} = 1.90 \text{ mm}^2$$

STATUS NG!

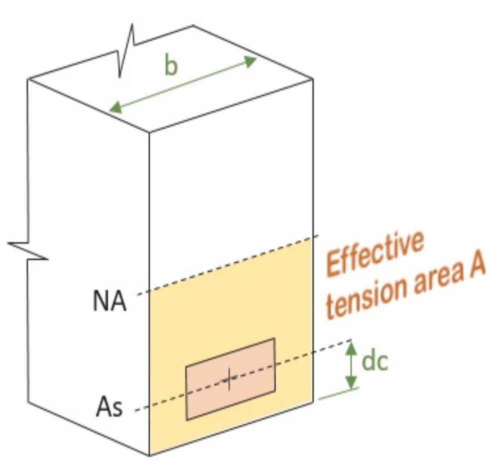
$$A_{st} = 2.37 \text{ mm}^2 \geq A_{st,min} = 0.47 \text{ mm}^2$$

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 = 16.67 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

$$w = 0.076 \cdot \beta_h \cdot f_s \cdot \sqrt[3]{d_c \cdot A} = 0.076 \cdot 1.2 \cdot 36000.00 \cdot \sqrt[3]{2.5 \cdot 16.67} = 0.0114 \text{ in}$$

3. Determine maximum code-permitted bar spacing, s

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

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

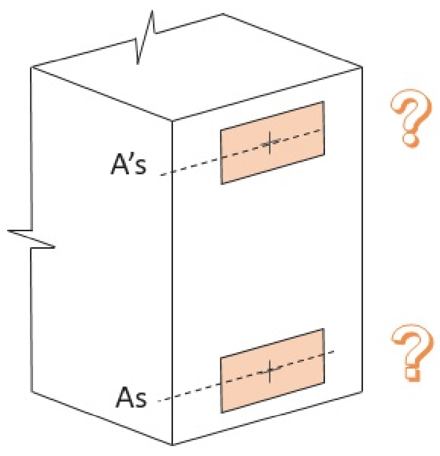
$$w = 0.0114 \text{ in} \leq w_{lim} = 0.012 \text{ in}$$

STATUS OK!

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

STATUS OK!

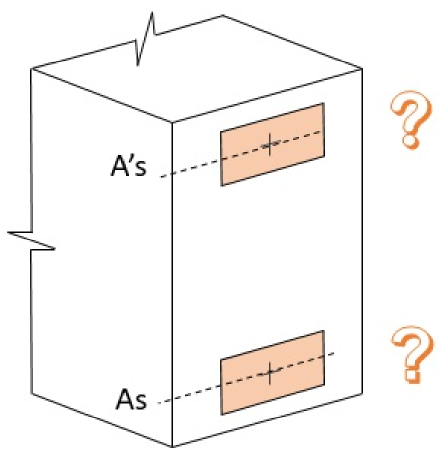
Flexure check (Negative bending moment case)



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)



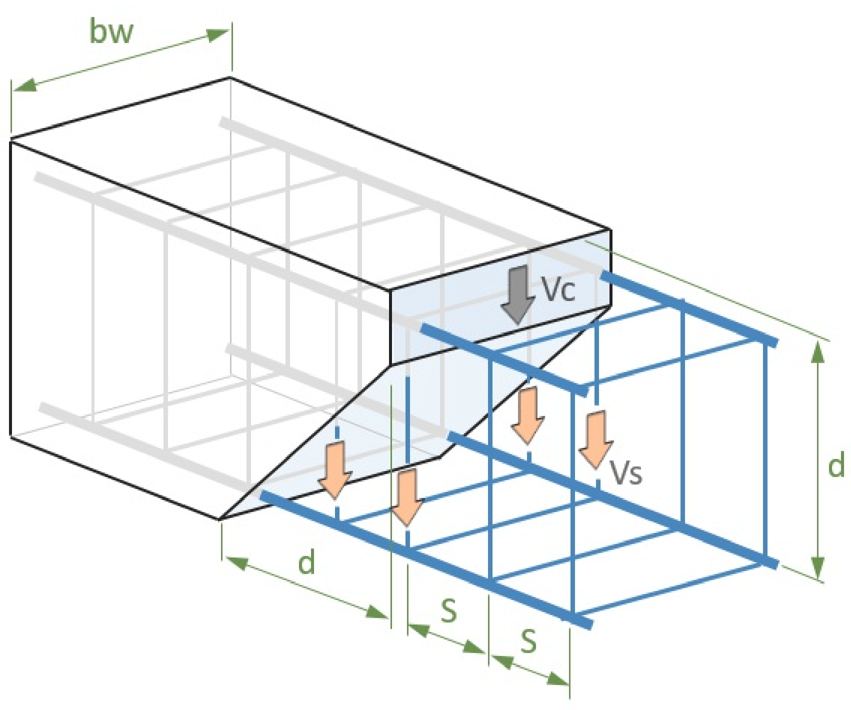
Bottom Reinforcement is absent in the section. Design checks can't be performed. But as acting moment value is equal to zero no need to check.

STATUS OK!

CHAPTER 11 (Section 11.2)

Shear check

BEAM ANALYSIS FOR SHEAR (IN PLANE)



Section input data:

Working depth of reinforcement d = 14 in
Spacing of the stirrups s = 10 in
Number of stirrups that are intersected by inclined section n = 1
Web width b_w = 10 in
Tensile reinforcement area A_s = 2.37 in^2
Sum of the cross sectional areas of the stirrup legs A_v = 0.39 in^2
Concrete area A_g = 165.00 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 = 47.99 kip-ft
Given shear force according to Load Combination (1.2DL+1.6LL) V_u = 0.00 kip

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

$$A_{v,min} = 0.75 \cdot \sqrt{f_c} \cdot \frac{b_w \cdot s}{f_y} = 0.75 \cdot \sqrt{3000} \cdot \frac{10 \cdot 10}{50000} = 0.08 \, \text{in}^2$$

$$A_{v,min} = 0.08 \, \text{in}^2 < \frac{50 \cdot b_w \cdot s}{f_y} = \frac{50 \cdot 10 \cdot 10}{50000} = 0.10 \, \text{in}^2 \rightarrow A_{v,min} = 0.10 \, \text{in}^2$$

$$A_v = 0.39 \, \text{in}^2 \geq A_{v,min} = 0.10 \, \text{in}^2 \rightarrow \text{ area of shear reinforcement is satisfied}$$

STATUS OK!

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

$$S_{max} = \min\left[\frac{d}{2}, 24\right] = 7.00 \, \text{in}$$

$$s = 10 \, \text{in} > S_{max} = 7.00 \, \text{in} \rightarrow \text{ spacing of stirrups is not satisfied}$$

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

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

$$= 2 \cdot 1 \cdot \sqrt{3000} \cdot 10 \cdot 14 = 15336.23 \, \text{lb} = 15.34 \, \text{kip}$$

case beam subjected to flexure and shear

$$\lambda = 1.0$$

$$\rho_w = \frac{A_s}{b_w \cdot d} = \frac{2.37}{10 \cdot 14} = 0.02$$

$$\frac{V_u \cdot d}{M_u} = \frac{0.00 \cdot 14}{47.99} = 0.00 \leq 1.0$$

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

$$= (1.9 \cdot 1 \cdot \sqrt{3000} + 2.5 \cdot 0.02 \cdot 0.00) \cdot 10 \cdot 14 = 14569.42 \, \text{lb} = 14.57 \, \text{kip}$$

$$V_c = 14569.42 \, \text{lb} = 14.57 \, \text{kip} \leq 3.5 \cdot \lambda \cdot \sqrt{f_c} \cdot b_w \cdot d =$$

$$= 3.5 \cdot 1 \cdot \sqrt{3000} \cdot 10 \cdot 14 = 26838.41 \, \text{lb} = 26.84 \, \text{kip}$$

$$V_c = 14569.42 \, \text{lb} = 14.57 \, \text{kip}$$

For design check V_c is taken as 14.57 kip

5. Calculate design resisting shear (V_R)

$$V_R = \phi \cdot (V_c + V_s) = 0.75 \cdot (14569.42 + 19634.95) = 25.65 \, \text{kip}$$

$$V_u = 0.00 \, \text{kip} \leq V_R = 25.65 \, \text{kip}$$

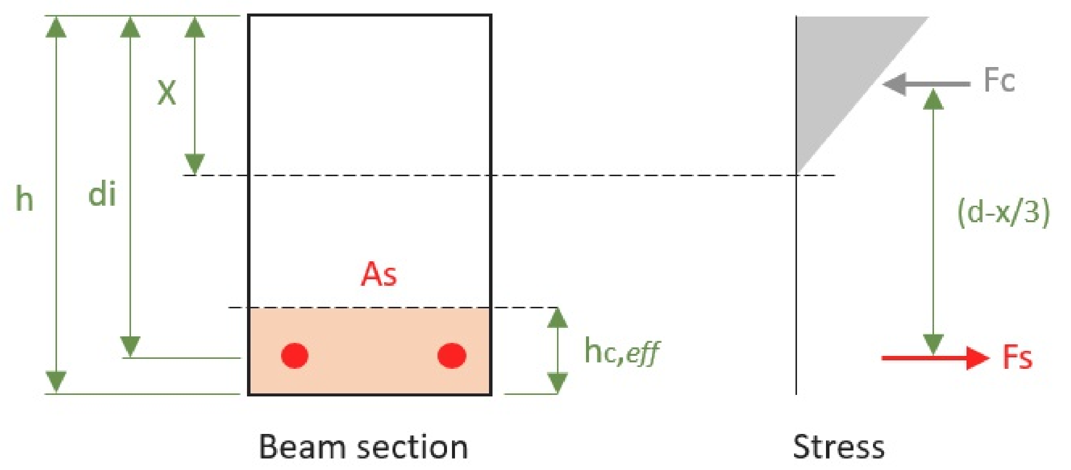
STATUS OK!

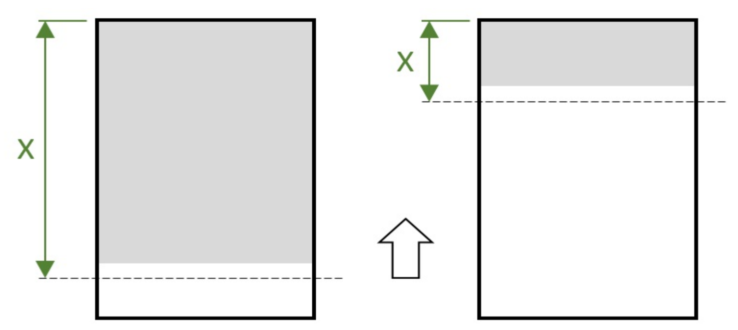
Deflection check

DEFLECTION OF BEAM (immediate and long-time)

Section input data:
Weight concrete factor $\lambda = 1$
Given service bending moment due to Dead Load $M_{a,DL} = 19.99$ Kip-ft
Given service bending moment due to Live Load $M_{a,LL} = 15.00$ Kip-ft
Typical dflection factor $k = 0.106$
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 $E_c = 57000 \cdot \sqrt{\acute{f_c}} = \sqrt{3000} = 3122018.58$ psi
Member span $L = 360$ in
Modular ratio $n = E_s/E_c = 9.29$

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 14 to 0 in)

Iter.	x (in)	As (in2)	Asc (in2)	Left force equil. part (lb)	Right force equil. part (lb)	Ratio
1	14.00	0.00	0.00	1288.20	2268.20	1.761
2	13.72	2.37	0.00	1249.40	2184.42	1.748
3	13.44	2.37	0.00	1211.37	2102.21	1.735
4	13.16	2.37	0.00	1174.13	2021.57	1.722
5	12.88	2.37	0.00	1137.68	1942.49	1.707
6	12.60	2.37	0.00	1102.00	1864.98	1.692
7	12.32	2.37	0.00	1067.12	1789.04	1.677
8	12.04	2.37	0.00	1033.01	1714.67	1.660
9	11.76	2.37	0.00	999.69	1641.87	1.642
10	11.48	2.37	0.00	967.16	1570.63	1.624
11	11.20	2.37	0.00	935.40	1500.96	1.605
12	10.92	2.37	0.00	904.44	1432.86	1.584
13	10.64	2.37	0.00	874.25	1366.33	1.563
14	10.36	2.37	0.00	844.85	1301.37	1.540
15	10.08	2.37	0.00	816.24	1237.97	1.517
16	9.80	2.37	0.00	788.40	1176.14	1.492

17	9.52	2.37	0.00	761.36	1115.88	1.466
18	9.24	2.37	0.00	735.09	1057.19	1.438
19	8.96	2.37	0.00	709.61	1000.07	1.409
20	8.68	2.37	0.00	684.92	944.51	1.379
21	8.40	2.37	0.00	661.00	890.52	1.347
22	8.12	2.37	0.00	637.88	838.10	1.314
23	7.84	2.37	0.00	615.53	787.25	1.279
24	7.56	2.37	0.00	593.97	737.97	1.242
25	7.28	2.37	0.00	573.20	690.25	1.204
26	7.00	2.37	0.00	553.20	644.10	1.164
27	6.72	2.37	0.00	534.00	599.52	1.123
28	6.44	2.37	0.00	515.57	556.51	1.079
29	6.16	2.37	0.00	497.93	515.07	1.034
left part < right part. Updating of iterations						
1	5.88	2.37	0.00	481.08	475.19	0.988
2	6.15	2.37	0.00	497.59	514.25	1.033
3	6.15	2.37	0.00	497.24	513.44	1.033
4	6.14	2.37	0.00	496.90	512.63	1.032
5	6.14	2.37	0.00	496.56	511.82	1.031
6	6.13	2.37	0.00	496.21	511.01	1.030
7	6.13	2.37	0.00	495.87	510.20	1.029
8	6.12	2.37	0.00	495.53	509.39	1.028
9	6.12	2.37	0.00	495.18	508.58	1.027
10	6.11	2.37	0.00	494.84	507.77	1.026
11	6.10	2.37	0.00	494.50	506.97	1.025
12	6.10	2.37	0.00	494.16	506.16	1.024
13	6.09	2.37	0.00	493.82	505.35	1.023
14	6.09	2.37	0.00	493.47	504.55	1.022
15	6.08	2.37	0.00	493.13	503.74	1.022
16	6.08	2.37	0.00	492.79	502.94	1.021
17	6.07	2.37	0.00	492.45	502.14	1.020
18	6.06	2.37	0.00	492.11	501.33	1.019
19	6.06	2.37	0.00	491.77	500.53	1.018
20	6.05	2.37	0.00	491.43	499.73	1.017
21	6.05	2.37	0.00	491.10	498.93	1.016
22	6.04	2.37	0.00	490.76	498.13	1.015
23	6.04	2.37	0.00	490.42	497.33	1.014
24	6.03	2.37	0.00	490.08	496.53	1.013
25	6.03	2.37	0.00	489.74	495.73	1.012
26	6.02	2.37	0.00	489.41	494.93	1.011
27	6.01	2.37	0.00	489.07	494.13	1.010
28	6.01	2.37	0.00	488.73	493.34	1.009
29	6.00	2.37	0.00	488.40	492.54	1.008
30	6.00	2.37	0.00	488.06	491.75	1.008

31	5.99	2.37	0.00	487.72	490.95	1.007
32	5.99	2.37	0.00	487.39	490.16	1.006
33	5.98	2.37	0.00	487.05	489.36	1.005
34	5.98	2.37	0.00	486.72	488.57	1.004
35	5.97	2.37	0.00	486.39	487.78	1.003
36	5.96	2.37	0.00	486.05	486.99	1.002
37	5.96	2.37	0.00	485.72	486.20	1.001
38	5.95	2.37	0.00	485.38	485.41	1.000
39	5.95	2.37	0.00	485.05	484.62	1.00

Value of x is $5.95\ \text{in}$
Tensioning rebar area $A_s = 2.37\ \text{in}^2$
Compression rebar area $A_{sc} = 0.00\ \text{in}^2$
Working depth of reinforcement $d = 14.00\ \text{in}$

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

$$I_g = \frac{b \cdot h^3}{12} = \frac{10 \cdot 16.5^3}{12} = 3743.44\ \text{in}^4$$

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

$$I_{cr} = \frac{b \cdot x^3}{3} + a_e \cdot A_s \cdot (d-x)^2 = \frac{10 \cdot 5.95^3}{3} + 9.29 \cdot 2.37 \cdot (14.00 - 5.95)^2 = 2128.75\ \text{in}^4$$

4. Moment that will cause cracking of the section:

$$f_r = 7.5 \cdot \lambda \cdot \sqrt{\acute{f_c}} = 7.5 \cdot 1.00 \cdot \sqrt{3000} = 410.79\ \text{psi}$$

$$M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{410.79 \cdot 0.001 \cdot 3743.44}{8.25 \cdot 12} = 15.53\ \text{Kip-ft}$$

5. Calculate the effective moment of inertia

$$I_e = \left(\frac{M_{cr}}{(M_{a,DL} + M_{a,LL})}\right)^3 \cdot I_g + \left[1 - \left(\frac{M_{cr}}{(M_{a,DL} + M_{a,LL})}\right)^3\right] \cdot I_{cr} = \left(\frac{15.53}{34.99}\right)^3 \cdot 3743.44 + \left[1 - \left(\frac{15.53}{34.99}\right)^3\right] \cdot 2128.75$$
$$= 2270.02\ \text{in}^4$$

$$I_e \leq I_g$$

6. Calculate the curvature of section

$$\text{Immediate Dead Load: } (1/r)_{e,DL} = \frac{M_a^*}{E_c \cdot I_e} = \frac{19.99 \cdot 12}{3122018.58 \cdot 0.001 \cdot 2270.02} = 0.0000338494\ /\text{in}$$

$$\text{Immediate Live Load: } (1/r)_{e,LL} = \frac{M_a^*}{E_c \cdot I_e} = \frac{15.00 \cdot 12}{3122018.58 \cdot 0.001 \cdot 2270.02} = 0.0000253968\ /\text{in}$$

7. Calculate the deflection based on section curvature

$$\text{Immediate Dead Load: } \triangle_{DL} = k \cdot L^2 \cdot (1/r)_{e,DL} = 0.106 \cdot 360^2 \cdot 0.00003385 = 0.4650\ \text{in}$$

$$\text{Immediate Live Load: } \triangle_{LL} = k \cdot L^2 \cdot (1/r)_{e,LL} = 0.106 \cdot 360^2 \cdot 0.00002540 = 0.3489\ \text{in}$$

$$\text{Immediate total: } \triangle = \triangle_{DL} + \triangle_{LL} = 0.4650 + 0.3489 = 0.81\ \text{in}$$

8. Calculate the additional long-time deflection

$$\acute{\rho} = \frac{\acute{A_s}}{b \cdot d} = \frac{0.000}{10 \cdot 14.00} = 0.00000\ \text{in}$$

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

	Long-time: $\triangle_{LT} = \lambda_{\triangle} \cdot (0.5 \cdot \lambda_{\triangle_{LL}} + \triangle_{DL}) + \triangle_{LL} = 2.00 \cdot (0.5 \cdot 0.35 + 0.47) + 0.35 = 1.6278 \text{ in}$ 9. Check with limited deflection $\triangle_{LL} = 0.3489 \text{ in} \leq \frac{L}{180} = \frac{360.00}{180} = 2.00 \text{ in}$ $\triangle_{LT} = 1.6278 \text{ in} > \frac{L}{240} = \frac{360.00}{240} = 1.50 \text{ in}$	**STATUS OK!** **STATUS NG!**