REFERENCES	CALCULATIONS	RESULTS

MEMBER #1 (SECTION POSITION 0.0 INCHES) BEAM DESIGN REPORT

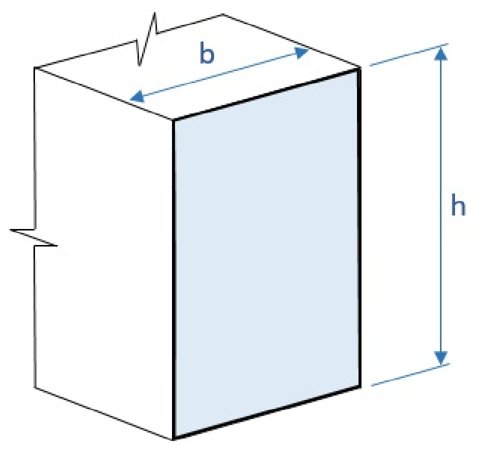
Code: ACI 318-14

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 $h = 35.5 \text{ in}$
Width $b = 18 \text{ in}$
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 $f_{yt} = 60000 \text{ psi}$
Permissible crack width $c_w = 0.012 \text{ in}$

Load Combinations (Ultimate Limit State)

For axial force in section:
LC1: USER = 0 Kip

For bending moment in section:
LC1: USER = 0 Kip-ft

For shear force in section:
LC1: USER = 0 Kip

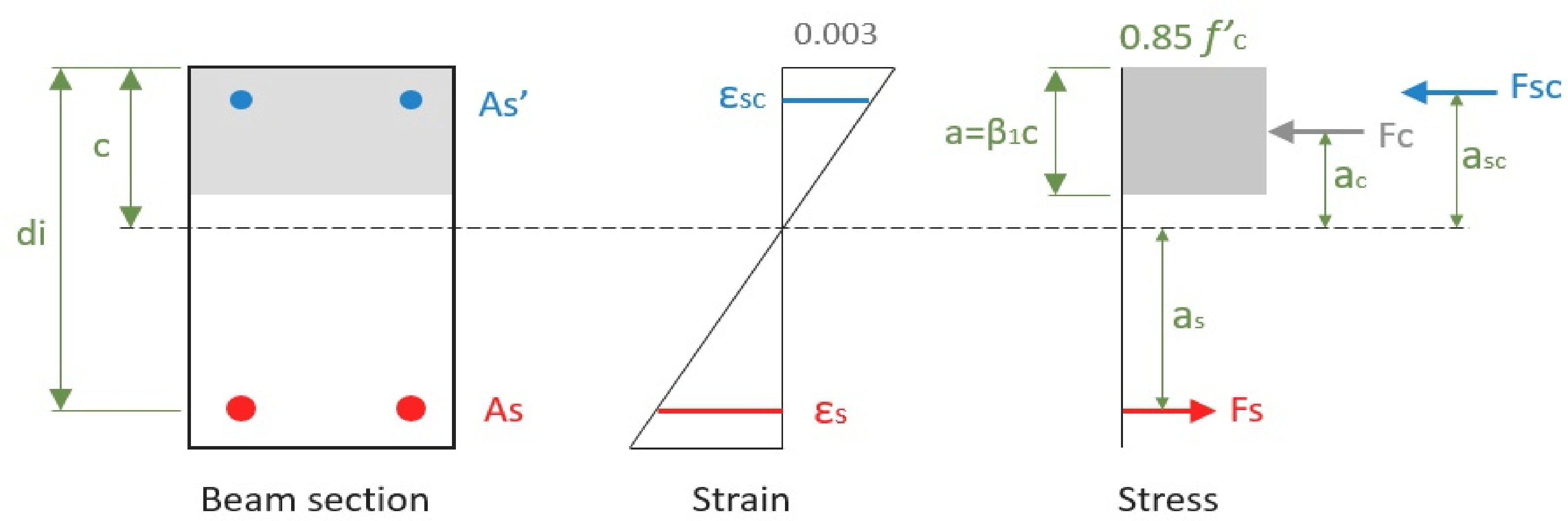
Load Combinations (Serviceability Limit State)

For bending moment in section:
LC1: USER = 0 Kip-ft

CHAPTER 9 (Section 9.5)

Flexure check (Positive bending moment case)

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 $d_t = 33 \text{ in}$
Given bending moment $M = 0.00 \text{ kip-ft}$

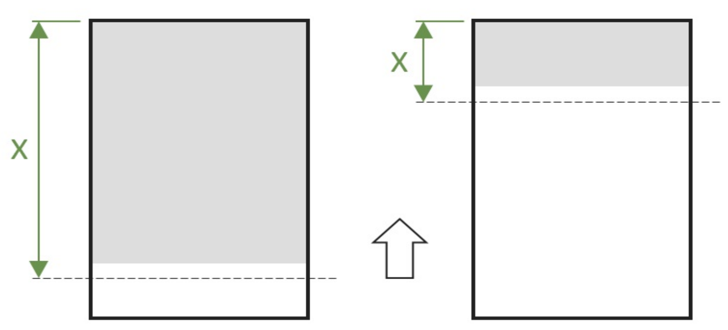
Section Rebar

Depth di (in)	bar diameter (in)	bar area Asi (in^2)
33.00	0.750	0.44
33.00	0.750	0.44
33.00	0.750	0.44
33.00	0.750	0.44
33.00	0.750	0.44
33.00	0.750	0.44
33.00	0.750	0.44

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 33 to 0 in)

Iter.	c (in)	c/dt	Fc (lbf)	Fcs (lbf)	Fc + Fcs (lbf)	Fs (lbf)	Ratio
1	33.0	1.00	1716660.00	0.00	1716660.00	0.00	Infinity
2	32.3	0.98	1682326.80	0.00	1682326.80	5468.57	307.636
3	31.7	0.96	1647993.60	0.00	1647993.60	11165.00	147.604
4	31.0	0.94	1613660.40	0.00	1613660.40	17103.83	94.345
5	30.4	0.92	1579327.20	0.00	1579327.20	23300.87	67.780
6	29.7	0.90	1544994.00	0.00	1544994.00	29773.33	51.892
7	29.0	0.88	1510660.80	0.00	1510660.80	36540.00	41.343
8	28.4	0.86	1476327.60	0.00	1476327.60	43621.40	33.844
9	27.7	0.84	1441994.40	0.00	1441994.40	51040.00	28.252
10	27.1	0.82	1407661.20	0.00	1407661.20	58820.49	23.931
11	26.4	0.80	1373328.00	0.00	1373328.00	66990.00	20.500
12	25.7	0.78	1338994.80	0.00	1338994.80	75578.46	17.717
13	25.1	0.76	1304661.60	0.00	1304661.60	84618.95	15.418
14	24.4	0.74	1270328.40	0.00	1270328.40	94148.11	13.493
15	23.8	0.72	1235995.20	0.00	1235995.20	104206.67	11.861
16	23.1	0.70	1201662.00	0.00	1201662.00	114840.00	10.464
17	22.4	0.68	1167328.80	0.00	1167328.80	126098.82	9.257
18	21.8	0.66	1132995.60	0.00	1132995.60	138040.00	8.208
19	21.1	0.64	1098662.40	0.00	1098662.40	150727.50	7.289

20	20.5	0.62	1064329.20	0.00	1064329.20	164233.55	6.481
21	19.8	0.60	1029996.00	0.00	1029996.00	178640.00	5.766
22	19.1	0.58	995662.80	0.00	995662.80	184800.00	5.388
23	18.5	0.56	961329.60	0.00	961329.60	184800.00	5.202
24	17.8	0.54	926996.40	0.00	926996.40	184800.00	5.016
25	17.2	0.52	892663.20	0.00	892663.20	184800.00	4.830
26	16.5	0.50	858330.00	0.00	858330.00	184800.00	4.645
27	15.8	0.48	823996.80	0.00	823996.80	184800.00	4.459
28	15.2	0.46	789663.60	0.00	789663.60	184800.00	4.273
29	14.5	0.44	755330.40	0.00	755330.40	184800.00	4.087
30	13.9	0.42	720997.20	0.00	720997.20	184800.00	3.901
31	13.2	0.40	686664.00	0.00	686664.00	184800.00	3.716
32	12.5	0.38	652330.80	0.00	652330.80	184800.00	3.530
33	11.9	0.36	617997.60	0.00	617997.60	184800.00	3.344
34	11.2	0.34	583664.40	0.00	583664.40	184800.00	3.158
35	10.6	0.32	549331.20	0.00	549331.20	184800.00	2.973
36	9.9	0.30	514998.00	0.00	514998.00	184800.00	2.787
37	9.2	0.28	480664.80	0.00	480664.80	184800.00	2.601
38	8.6	0.26	446331.60	0.00	446331.60	184800.00	2.415
39	7.9	0.24	411998.40	0.00	411998.40	184800.00	2.229
40	7.3	0.22	377665.20	0.00	377665.20	184800.00	2.044
41	6.6	0.20	343332.00	0.00	343332.00	184800.00	1.858
42	5.9	0.18	308998.80	0.00	308998.80	184800.00	1.672
43	5.3	0.16	274665.60	0.00	274665.60	184800.00	1.486
44	4.6	0.14	240332.40	0.00	240332.40	184800.00	1.300
45	4.0	0.12	205999.20	0.00	205999.20	184800.00	1.115
(Fc + Fcs) < Fs. Updating of iterations							
1	3.3	0.10	171666.00	0.00	171666.00	184800.00	0.929
2	3.9	0.12	205312.54	0.00	205312.54	184800.00	1.111
3	3.9	0.12	204625.87	0.00	204625.87	184800.00	1.107
4	3.9	0.12	203939.21	0.00	203939.21	184800.00	1.104
5	3.9	0.12	203252.54	0.00	203252.54	184800.00	1.100
6	3.9	0.12	202565.88	0.00	202565.88	184800.00	1.096
7	3.9	0.12	201879.22	0.00	201879.22	184800.00	1.092
8	3.9	0.12	201192.55	0.00	201192.55	184800.00	1.089
9	3.9	0.12	200505.89	0.00	200505.89	184800.00	1.085
10	3.8	0.12	199819.22	0.00	199819.22	184800.00	1.081
11	3.8	0.12	199132.56	0.00	199132.56	184800.00	1.078
12	3.8	0.12	198445.90	0.00	198445.90	184800.00	1.074
13	3.8	0.12	197759.23	0.00	197759.23	184800.00	1.070
14	3.8	0.11	197072.57	0.00	197072.57	184800.00	1.066
15	3.8	0.11	196385.90	0.00	196385.90	184800.00	1.063
16	3.8	0.11	195699.24	0.00	195699.24	184800.00	1.059
17	3.7	0.11	195012.58	0.00	195012.58	184800.00	1.055

18	3.7	0.11	194325.91	0.00	194325.91	184800.00	1.052
19	3.7	0.11	193639.25	0.00	193639.25	184800.00	1.048
20	3.7	0.11	192952.58	0.00	192952.58	184800.00	1.044
21	3.7	0.11	192265.92	0.00	192265.92	184800.00	1.040
22	3.7	0.11	191579.26	0.00	191579.26	184800.00	1.037
23	3.7	0.11	190892.59	0.00	190892.59	184800.00	1.033
24	3.7	0.11	190205.93	0.00	190205.93	184800.00	1.029
25	3.6	0.11	189519.26	0.00	189519.26	184800.00	1.026
26	3.6	0.11	188832.60	0.00	188832.60	184800.00	1.022
27	3.6	0.11	188145.94	0.00	188145.94	184800.00	1.018
28	3.6	0.11	187459.27	0.00	187459.27	184800.00	1.014
29	3.6	0.11	186772.61	0.00	186772.61	184800.00	1.011
30	3.6	0.11	186085.94	0.00	186085.94	184800.00	1.007
31	3.6	0.11	185399.28	0.00	185399.28	184800.00	1.003
32	3.55	0.11	184712.62	0.00	184712.62	184800.00	1.000

Final value of c is 3.55 in and flexural tension reinforcement area is 3.08 in^2
Working depth of reinforcement $d = 33.00 \text{ in}$

$$e_t = \frac{d_t - c}{c} \cdot (0.003) = \frac{33 - 3.55}{3.55} \cdot (0.003) = 0.02488 \geq 0.005 \rightarrow \phi = 0.9$$

2. Calculation moment resistance M_R

$$M_R = \phi \cdot M = \phi \cdot (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) = 0.90 \cdot (377129.60 + 0.00 + 5442212.16) = 5237.41 \text{ Ibf-in} = 436.45 \text{ Kip-ft}$$

$$M = 0.00 \text{ Kip-ft} \leq M_R = 436.45 \text{ Kip-ft}$$

STATUS OK!

3. Minimum required flexural tension reinforcement in a beam section

$$A_{s,min} = \frac{3 \cdot \sqrt{f_c}}{f_y} \cdot b_w \cdot d = \frac{3 \cdot \sqrt{4000}}{60000} \cdot 18 \cdot 33.00 = 1.88 \text{ in}^2$$

$$A_{s,min} \leq \frac{200}{f_y} \cdot b_w \cdot d = \frac{200}{60000} \cdot 18 \cdot 33.00 = 1.98 \text{ in}^2 \rightarrow A_{s,min} = 1.98 \text{ in}^2$$

4. Maximum required flexural tension reinforcement in a beam section

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

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

$$A_{s,max} = \rho_{max} \cdot b_w \cdot d = 0.01806 \cdot 18 \cdot 33.00 = 10.73 \text{ in}^2$$

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

$$A_{st} = 3.08 \text{ mm}^2 \leq A_{st,max} = 10.73 \text{ mm}^2$$

STATUS OK!

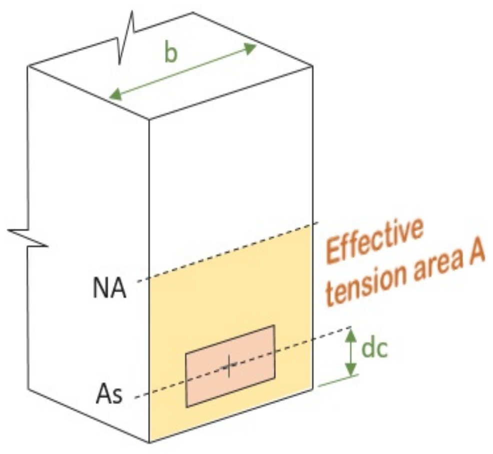
$$A_{st} = 3.08 \text{ mm}^2 \geq A_{st,min} = 1.98 \text{ mm}^2$$

STATUS OK!

Crack check (Positive bending moment case)

CHAPTER 10 (Section 10.6)

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 = 12.86 in^2
Clear cover c_c = 2.125 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

$$w = 0.076 \cdot \beta_h \cdot f_s \cdot \sqrt[3]{d_c \cdot A} = 0.076 \cdot 1.2 \cdot 36000.00 \cdot \sqrt[3]{2.5 \cdot 12.86} = 0.0104 \text{ in}$$

3. Determine maximum code-permitted bar spacing, s

$$s = 15 \cdot (\frac{40000}{f_s}) - 2.5 \cdot c_c = 15 \cdot (\frac{40000}{36000.00}) - 2.5 \cdot 2.125 = 11.35 \text{ in}$$

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

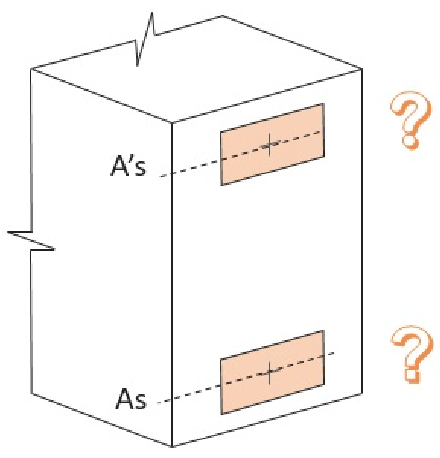
$$w = 0.0104 \text{ in} \leq w_{lim} = 0.012 \text{ in}$$

STATUS OK!

$$c_c = 2.125 \text{ in} \leq s = 11.35 \text{ in}$$

STATUS OK!

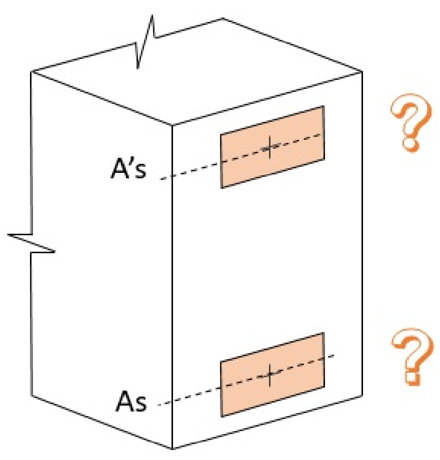
Flexure check (Negative bending moment case)



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)



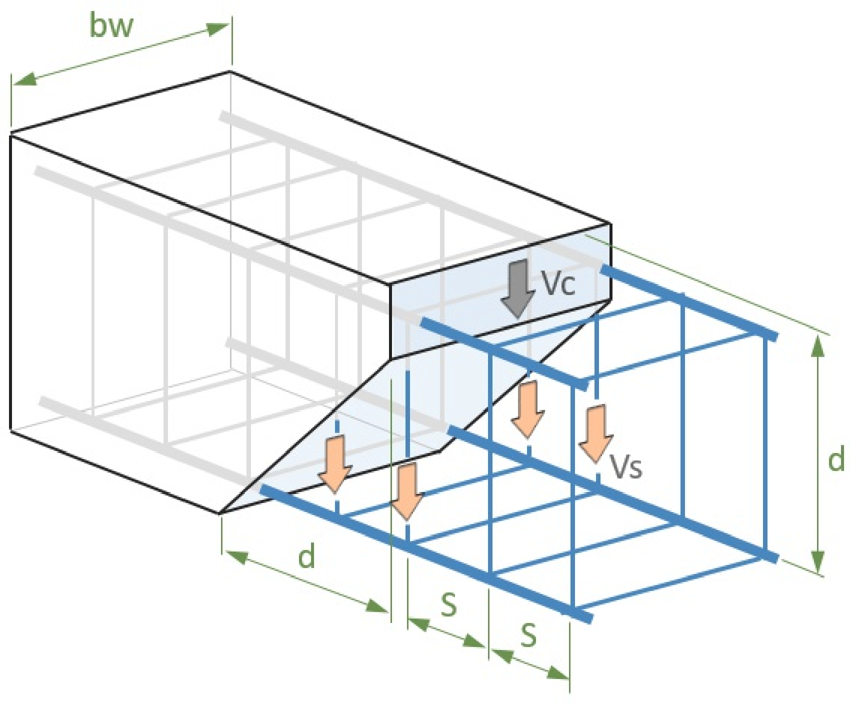
Bottom Reinforcement is absent in the section. Design checks can't be performed. But as acting moment value is equal to zero no need to check.

STATUS OK!

CHAPTER 11 (Section 11.2)

Shear check

BEAM ANALYSIS FOR SHEAR (IN PLANE)



Section input data:
Working depth of reinforcement d = 33 in
Spacing of the stirrups s = 12 in
Number of stirrups that are intersected by inclined section n = 3
Web width b_w = 18 in
Tensile reinforcement area A_s = 3.08 in^2
Sum of the cross sectional areas of the stirrup legs A_v = 0.22 in^2
Concrete area A_g = 639.00 in^2
Given axial force according to Load Combination (USER) N_u = 0.00 kip
Given bending moment according to Load Combination (USER) M_u = 0.00 kip-ft
Given shear force according to Load Combination (USER) V_u = 0.00 kip

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

$$A_{v,min} = 0.75 \cdot \sqrt{f_c} \cdot \frac{b_w \cdot s}{f_y} = 0.75 \cdot \sqrt{4000} \cdot \frac{18 \cdot 12}{60000} = 0.17\,\text{in}^2$$

$$A_{v,min} = 0.17\,\text{in}^2 < \frac{50 \cdot b_w \cdot s}{f_y} = \frac{50 \cdot 18 \cdot 12}{60000} = 0.18\,\text{in}^2 \rightarrow A_{v,min} = 0.18\,\text{in}^2$$

$$A_v = 0.22\,\text{in}^2 \geq A_{v,min} = 0.18\,\text{in}^2 \rightarrow \text{ area of shear reinforcement is satisfied}$$

STATUS OK!

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

$$S_{max} = \min\left[\frac{d}{2}, 24\right] = 16.50\,\text{in}$$

$$s = 12\,\text{in} \leq S_{max} = 16.50\,\text{in} \rightarrow \text{ spacing of stirrups is satisfied}$$

STATUS OK!

3. Calculate shear strength of section stirrups (V_s)

$$V_s = n \cdot A_v \cdot f_y = 3 \cdot 0.22 \cdot 60000 = 39760.78\,\text{lb} = 39.76\,\text{kip}$$

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

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

$$= 2 \cdot 1 \cdot \sqrt{4000} \cdot 18 \cdot 33 = 75135.72\,\text{lb} = 75.14\,\text{kip}$$

For design check V_c is taken as $75.14\,\text{kip}$

5. Calculate design resisting shear (V_R)

$$V_R = \phi \cdot (V_c + V_s) = 0.75 \cdot (75135.72 + 39760.78) = 86.17\,\text{kip}$$

$$V_u = 0.00\,\text{kip} \leq V_R = 86.17\,\text{kip}$$

STATUS OK!