REFERENCES	CALCULATIONS	RESULTS

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

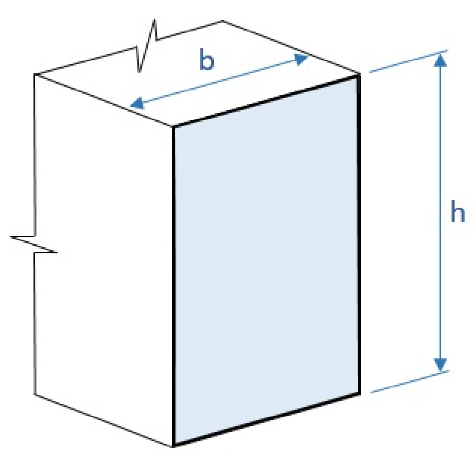
Code: ACI 318-14

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 $h = 12.75 \text{ in}$
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 $f_{yt} = 50000 \text{ psi}$
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

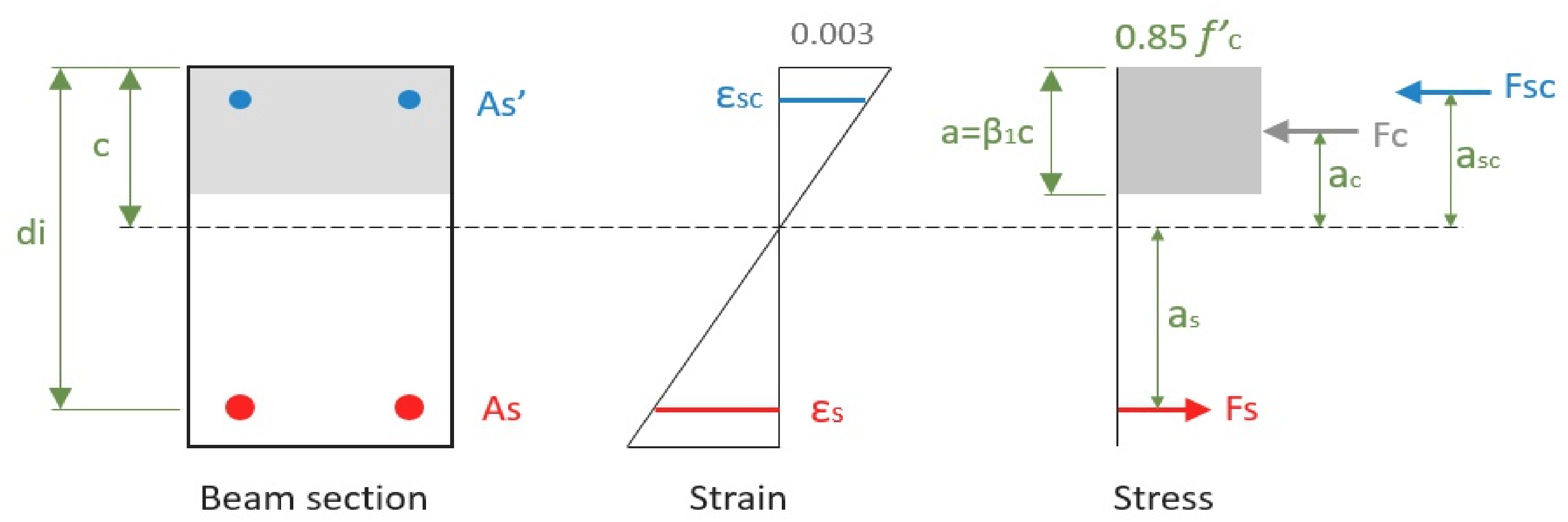
Load Combinations (Serviceability Limit State)

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

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 $d_t = 10.25 \text{ in}$
Given bending moment $M = 0.00 \text{ kip-ft}$

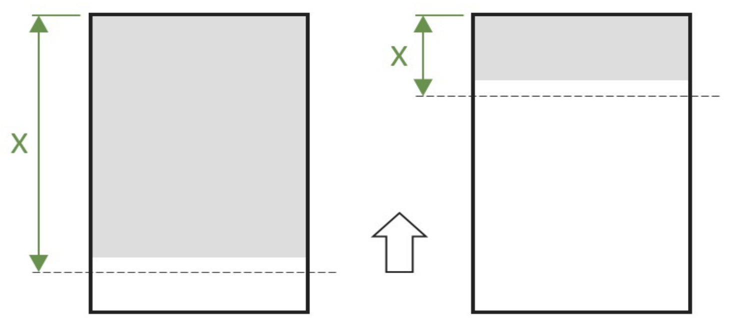
Section Rebar

Depth di (in)	bar diameter (in)	bar area Asi (in^2)
10.25	0.500	0.20
10.25	0.500	0.20
10.25	0.500	0.20
10.25	0.500	0.20
10.25	0.500	0.20

Rectangular compression block factor

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

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 $f_s = \{e_s E_s \ (e_s \leq e_y), \ e_y \ (e_s > e_y)\}$
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 10.25 to 0 in)

Iter.	c (in)	c/dt	Fc (lbf)	Fcs (lbf)	Fc + Fcs (lbf)	Fs (lbf)	Ratio
1	10.3	1.00	533205.00	0.00	533205.00	0.00	Infinity
2	10.0	0.98	522540.90	0.00	522540.90	1775.51	294.305
3	9.8	0.96	511876.80	0.00	511876.80	3625.00	141.207
4	9.6	0.94	501212.70	0.00	501212.70	5553.19	90.257
5	9.4	0.92	490548.60	0.00	490548.60	7565.22	64.843
6	9.2	0.90	479884.50	0.00	479884.50	9666.67	49.643
7	9.0	0.88	469220.40	0.00	469220.40	11863.64	39.551
8	8.8	0.86	458556.30	0.00	458556.30	14162.79	32.378
9	8.6	0.84	447892.20	0.00	447892.20	16571.43	27.028
10	8.4	0.82	437228.10	0.00	437228.10	19097.56	22.894
11	8.2	0.80	426564.00	0.00	426564.00	21750.00	19.612
12	8.0	0.78	415899.90	0.00	415899.90	24538.46	16.949
13	7.8	0.76	405235.80	0.00	405235.80	27473.68	14.750
14	7.6	0.74	394571.70	0.00	394571.70	30567.57	12.908
15	7.4	0.72	383907.60	0.00	383907.60	33833.33	11.347
16	7.2	0.70	373243.50	0.00	373243.50	37285.71	10.010
17	7.0	0.68	362579.40	0.00	362579.40	40941.18	8.856
18	6.8	0.66	351915.30	0.00	351915.30	44818.18	7.852
19	6.6	0.64	341251.20	0.00	341251.20	48937.50	6.973
20	6.4	0.62	330587.10	0.00	330587.10	53322.58	6.200
21	6.1	0.60	319923.00	0.00	319923.00	58000.00	5.516

22	5.9	0.58	309258.90	0.00	309258.90	60000.00	5.154
23	5.7	0.56	298594.80	0.00	298594.80	60000.00	4.977
24	5.5	0.54	287930.70	0.00	287930.70	60000.00	4.799
25	5.3	0.52	277266.60	0.00	277266.60	60000.00	4.621
26	5.1	0.50	266602.50	0.00	266602.50	60000.00	4.443
27	4.9	0.48	255938.40	0.00	255938.40	60000.00	4.266
28	4.7	0.46	245274.30	0.00	245274.30	60000.00	4.088
29	4.5	0.44	234610.20	0.00	234610.20	60000.00	3.910
30	4.3	0.42	223946.10	0.00	223946.10	60000.00	3.732
31	4.1	0.40	213282.00	0.00	213282.00	60000.00	3.555
32	3.9	0.38	202617.90	0.00	202617.90	60000.00	3.377
33	3.7	0.36	191953.80	0.00	191953.80	60000.00	3.199
34	3.5	0.34	181289.70	0.00	181289.70	60000.00	3.021
35	3.3	0.32	170625.60	0.00	170625.60	60000.00	2.844
36	3.1	0.30	159961.50	0.00	159961.50	60000.00	2.666
37	2.9	0.28	149297.40	0.00	149297.40	60000.00	2.488
38	2.7	0.26	138633.30	0.00	138633.30	60000.00	2.311
39	2.5	0.24	127969.20	0.00	127969.20	60000.00	2.133
40	2.3	0.22	117305.10	0.00	117305.10	60000.00	1.955
41	2.0	0.20	106641.00	0.00	106641.00	60000.00	1.777
42	1.8	0.18	95976.90	0.00	95976.90	60000.00	1.600
43	1.6	0.16	85312.80	0.00	85312.80	60000.00	1.422
44	1.4	0.14	74648.70	0.00	74648.70	60000.00	1.244
45	1.2	0.12	63984.60	0.00	63984.60	60000.00	1.066
(Fc + Fcs) < Fs. Updating of iterations							
1	1.0	0.10	53320.50	0.00	53320.50	60000.00	0.889
2	1.2	0.12	63771.32	0.00	63771.32	60000.00	1.063
3	1.2	0.12	63558.04	0.00	63558.04	60000.00	1.059
4	1.2	0.12	63344.75	0.00	63344.75	60000.00	1.056
5	1.2	0.12	63131.47	0.00	63131.47	60000.00	1.052
6	1.2	0.12	62918.19	0.00	62918.19	60000.00	1.049
7	1.2	0.12	62704.91	0.00	62704.91	60000.00	1.045
8	1.2	0.12	62491.63	0.00	62491.63	60000.00	1.042
9	1.2	0.12	62278.34	0.00	62278.34	60000.00	1.038
10	1.2	0.12	62065.06	0.00	62065.06	60000.00	1.034
11	1.2	0.12	61851.78	0.00	61851.78	60000.00	1.031
12	1.2	0.12	61638.50	0.00	61638.50	60000.00	1.027
13	1.2	0.12	61425.22	0.00	61425.22	60000.00	1.024
14	1.2	0.11	61211.93	0.00	61211.93	60000.00	1.020
15	1.2	0.11	60998.65	0.00	60998.65	60000.00	1.017
16	1.2	0.11	60785.37	0.00	60785.37	60000.00	1.013
17	1.2	0.11	60572.09	0.00	60572.09	60000.00	1.010
18	1.2	0.11	60358.81	0.00	60358.81	60000.00	1.006
19	1.2	0.11	60145.52	0.00	60145.52	60000.00	1.002

20	1.15	0.11	59932.24	0.00	59932.24	60000.00	0.999

Final value of c is 1.15 in and flexural tension reinforcement area is 1.00 in^2
Working depth of reinforcement $d = 10.25 \text{ in}$

$$e_t = \frac{d_t - c}{c} \cdot (0.003) = \frac{10.25 - 1.15}{1.15} \cdot (0.003) = 0.02369 \geq 0.005 \rightarrow \phi = 0.9$$

2. Calculation moment resistance M_R

$$M_R = \phi \cdot M = \phi \cdot (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) = 0.90 \cdot (39702.56 + 0.00 + 545874.00) = 527.02 \text{ Ibf-in} \\ = 43.92 \text{ Kip-ft}$$

$$M = 0.00 \text{ Kip-ft} \leq M_R = 43.92 \text{ Kip-ft}$$

STATUS OK!

3. Minimum required flexural tension reinforcement in a beam section

$$A_{s,min} = \frac{3 \cdot \sqrt{f_c}}{f_y} \cdot b_w \cdot d = \frac{3 \cdot \sqrt{4000}}{60000} \cdot 18 \cdot 10.25 = 0.58 \text{ in}^2$$

$$A_{s,min} \leq \frac{200}{f_y} \cdot b_w \cdot d = \frac{200}{60000} \cdot 18 \cdot 10.25 = 0.62 \text{ in}^2 \rightarrow A_{s,min} = 0.62 \text{ in}^2$$

4. Maximum required flexural tension reinforcement in a beam section

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

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

$$A_{s,max} = \rho_{max} \cdot b_w \cdot d = 0.01806 \cdot 18 \cdot 10.25 = 3.33 \text{ in}^2$$

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

$$A_{st} = 1.00 \text{ mm}^2 \leq A_{st,max} = 3.33 \text{ mm}^2$$

STATUS OK!

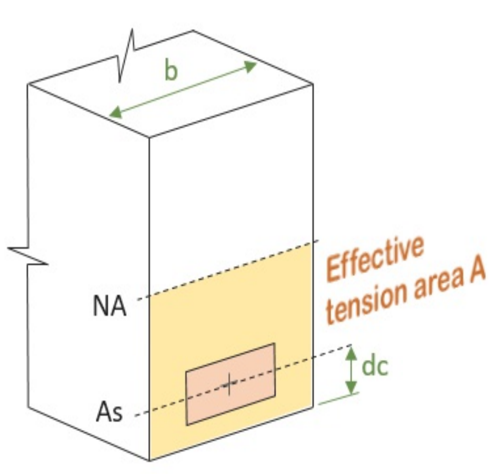
$$A_{st} = 1.00 \text{ mm}^2 \geq A_{st,min} = 0.62 \text{ mm}^2$$

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 = 18.00 in^2
Clear cover c_c = 2.25 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

$$w = 0.076 \cdot \beta_h \cdot f_s \cdot \sqrt[3]{d_c \cdot A} = 0.076 \cdot 1.2 \cdot 36000.00 \cdot \sqrt[3]{2.5 \cdot 18.00} = 0.0117 \text{ in}$$

3. Determine maximum code-permitted bar spacing, s

$$s = 15 \cdot (\frac{40000}{f_s}) - 2.5 \cdot c_c = 15 \cdot (\frac{40000}{36000.00}) - 2.5 \cdot 2.25 = 11.04 \text{ in}$$

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

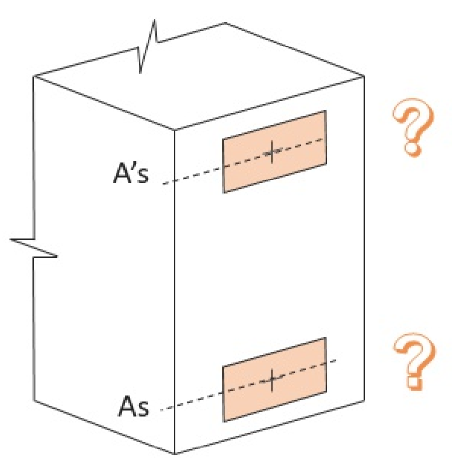
$$w = 0.0117 \text{ in} \le w_{lim} = 0.012 \text{ in}$$

STATUS OK!

$$c_c = 2.25 \text{ in} \le s = 11.04 \text{ in}$$

STATUS OK!

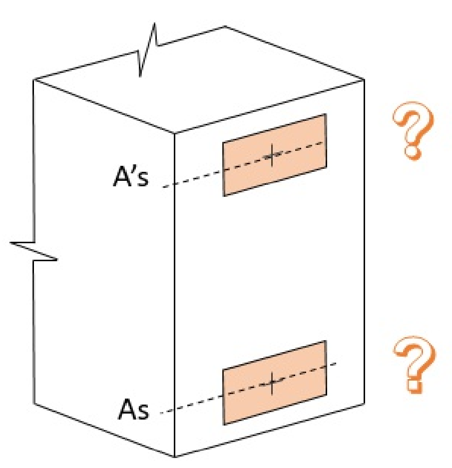
Flexure check (Negative bending moment case)



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)



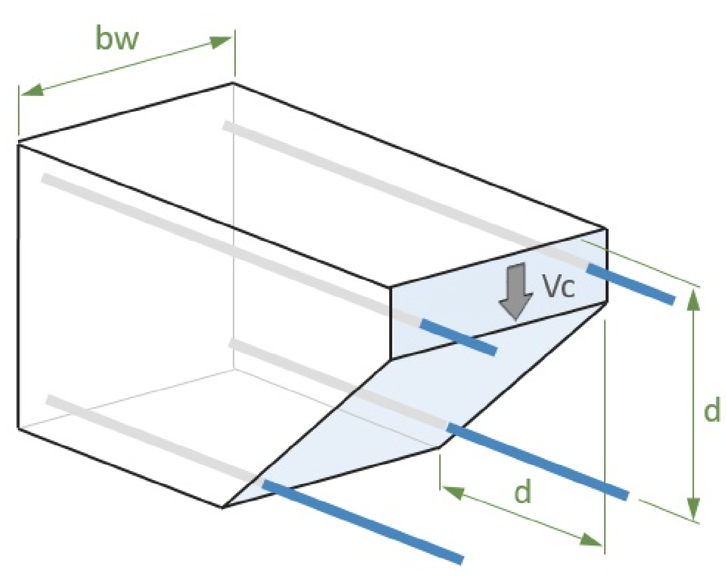
Bottom Reinforcement is absent in the section. Design checks can't be performed. But as acting moment value is equal to zero no need to check.

STATUS OK!

CHAPTER 11 (Section 11.2)

Shear check

BEAM ANALYSIS FOR SHEAR WITHOUT STIRRUPS (IN PLANE)



Section input data:
Web width b_w = 18 in
Working depth of reinforcement d = 10.25 in
Given shear force according to Load Combination (USER) V_u = 0.00 kip

Calculate design resisting shear (V_R)

$$V_R = 0.5 \cdot 0.75 \cdot 2 \cdot \sqrt{f_c} \cdot b_w \cdot d =$$

	$= 0.5 \cdot 0.75 \cdot 2 \cdot 4000 \cdot 18 \cdot 10.25 = 8751.60 \text{ lb } = 8.75 \text{ kip}$ $V_u = 0.00 \text{ kip} \leq V_R = 8.75 \text{ kip}$	**STATUS OK!**	