REFERENCES	CALCULATIONS	RESULTS

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

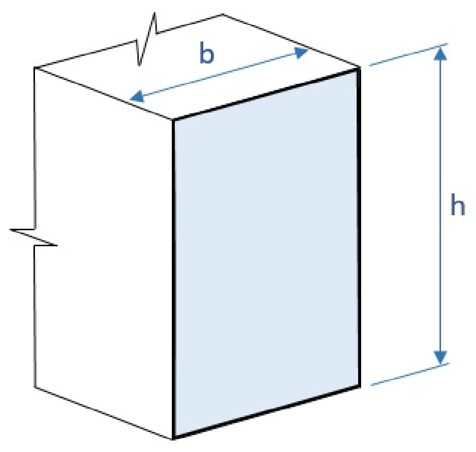
Code: ACI 318-14

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 $h = 23.6 \text{ in}$
Width $b = 11 \text{ in}$
Member length = 240 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)

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

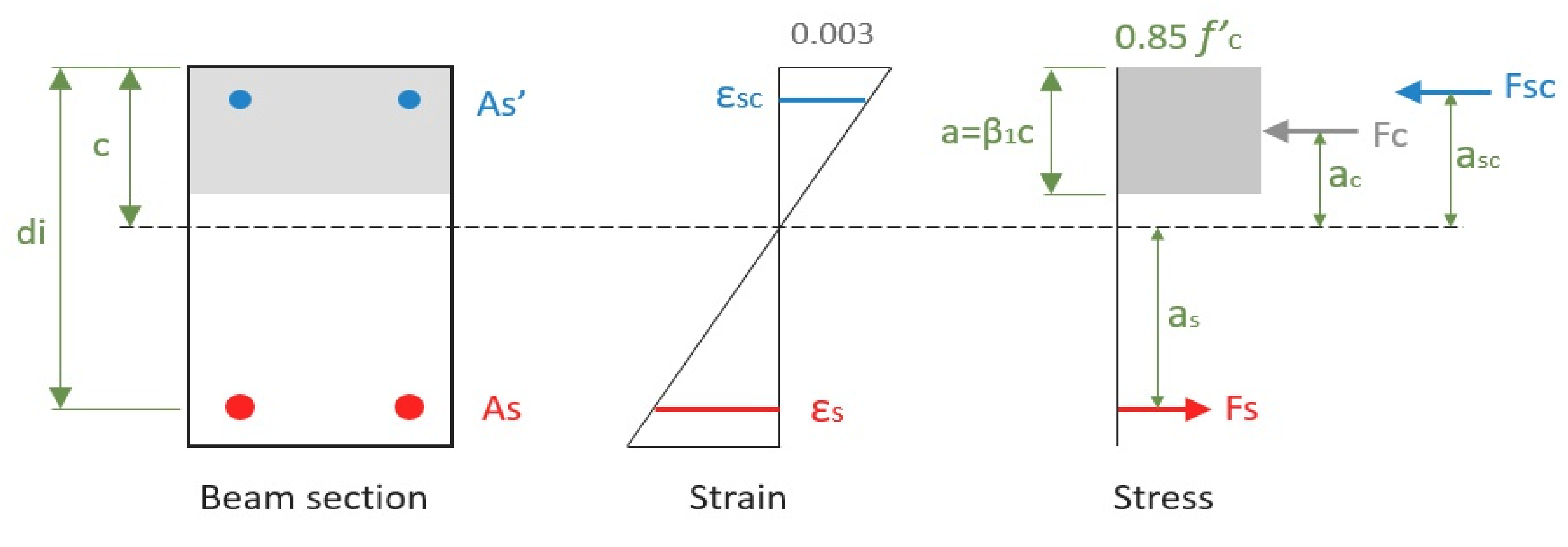
Load Combinations (Serviceability Limit State)

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 $d_t = 21.1 \text{ in}$
Given bending moment $M = 0.00 \text{ kip-ft}$

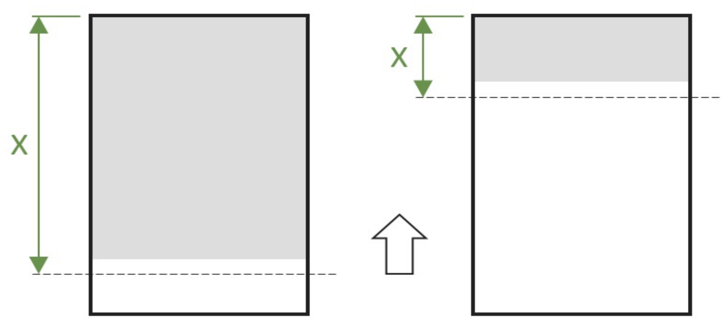
Section Rebar

Depth di (in)	bar diameter (in)	bar area Asi (in^2)
21.10	1.128	1.00
21.10	1.128	1.00
21.10	1.128	1.00
18.90	1.128	1.00
18.90	1.128	1.00
18.90	1.128	1.00
2.50	1.270	1.27
2.50	1.270	1.27

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

Iter.	c (in)	c/dt	Fc (lbf)	Fcs (lbf)	Fc + Fcs (lbf)	Fs (lbf)	Ratio
1	21.1	1.00	503076.75	179613.27	682690.02	0.00	Infinity
2	20.7	0.98	493015.22	174842.11	667857.33	5326.53	125.383
3	20.3	0.96	482953.68	169872.16	652825.84	10875.00	60.030
4	19.8	0.94	472892.14	164690.71	637582.86	16659.57	38.271
5	19.4	0.92	462830.61	159283.99	622114.60	22695.65	27.411
6	19.0	0.90	452769.07	153636.97	606406.04	29000.00	20.911
7	18.6	0.88	442707.54	152400.00	595107.54	40257.65	14.782
8	18.1	0.86	432646.00	152400.00	585046.00	53333.41	10.970
9	17.7	0.84	422584.47	152400.00	574984.47	67031.82	8.578
10	17.3	0.82	412522.93	152400.00	564922.93	81398.45	6.940
11	16.9	0.80	402461.40	152400.00	554861.40	96483.41	5.751
12	16.5	0.78	392399.86	152400.00	544799.86	112341.96	4.849
13	16.0	0.76	382338.33	152400.00	534738.33	129035.17	4.144
14	15.6	0.74	372276.79	152400.00	524676.79	146630.72	3.578
15	15.2	0.72	362215.26	152400.00	514615.26	165203.79	3.115
16	14.8	0.70	352153.72	152400.00	504553.72	184838.19	2.730
17	14.3	0.68	342092.19	152400.00	494492.19	205627.54	2.405
18	13.9	0.66	332030.65	152400.00	484430.65	227676.86	2.128
19	13.5	0.64	321969.12	152400.00	474369.12	251104.27	1.889

20	13.1	0.62	311907.58	152400.00	464307.58	276043.11	1.682
21	12.7	0.60	301846.05	152400.00	454246.05	302644.55	1.501
22	12.2	0.58	291784.51	152400.00	444184.51	322080.57	1.379
23	11.8	0.56	281722.98	152400.00	434122.98	336476.30	1.290
24	11.4	0.54	271661.44	152400.00	424061.44	351938.39	1.205
25	11.0	0.52	261599.91	152400.00	413999.91	360000.00	1.150
26	10.5	0.50	251538.37	152400.00	403938.37	360000.00	1.122
27	10.1	0.48	241476.84	152400.00	393876.84	360000.00	1.094
28	9.7	0.46	231415.30	152400.00	383815.30	360000.00	1.066
29	9.3	0.44	221353.77	152400.00	373753.77	360000.00	1.038
30	8.9	0.42	211292.23	152400.00	363692.23	360000.00	1.010
(Fc + Fcs) < Fs. Updating of iterations							
1	8.4	0.40	201230.70	152400.00	353630.70	360000.00	0.982
2	8.9	0.42	211091.00	152400.00	363491.00	360000.00	1.010
3	8.8	0.42	210889.77	152400.00	363289.77	360000.00	1.009
4	8.8	0.42	210688.54	152400.00	363088.54	360000.00	1.009
5	8.8	0.42	210487.31	152400.00	362887.31	360000.00	1.008
6	8.8	0.42	210286.08	152400.00	362686.08	360000.00	1.007
7	8.8	0.42	210084.85	152400.00	362484.85	360000.00	1.007
8	8.8	0.42	209883.62	152400.00	362283.62	360000.00	1.006
9	8.8	0.42	209682.39	152400.00	362082.39	360000.00	1.006
10	8.8	0.42	209481.16	152400.00	361881.16	360000.00	1.005
11	8.8	0.42	209279.93	152400.00	361679.93	360000.00	1.005
12	8.8	0.42	209078.70	152400.00	361478.70	360000.00	1.004
13	8.8	0.42	208877.47	152400.00	361277.47	360000.00	1.004
14	8.8	0.41	208676.24	152400.00	361076.24	360000.00	1.003
15	8.7	0.41	208475.01	152400.00	360875.01	360000.00	1.002
16	8.7	0.41	208273.77	152400.00	360673.77	360000.00	1.002
17	8.7	0.41	208072.54	152400.00	360472.54	360000.00	1.001
18	8.7	0.41	207871.31	152400.00	360271.31	360000.00	1.001
19	8.7	0.41	207670.08	152400.00	360070.08	360000.00	1.000
20	8.70	0.41	207468.85	152400.00	359868.85	360000.00	1.000

Final value of c is 8.70 in and flexural tension reinforcement area is 6.00 in^2
Working depth of reinforcement $d = 20.00 \text{ in}$

$$e_t = \frac{d_t - c}{c} \cdot (0.003) = \frac{21.1 - 8.70}{8.70} \cdot (0.003) = 0.00427$$

$$e_y < e_t < 0.005$$

$$\phi = 0.65 + (\frac{0.9 - 0.65}{0.005 - e_y}) \cdot (e_t - e_y) = 0.65 + (\frac{0.9 - 0.65}{0.005 - 0.00207}) \cdot (0.00427 - 0.00207) = 0.84$$

2. Calculation moment resistance M_R

$$M_R = \phi \cdot M = \phi \cdot (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) = 0.84 \cdot (1038058.57 + 945129.94 + 4067409.60) = 5071.12 \text{ Ibf-in} = 422.59 \text{ Kip-ft}$$

$$M = 0.00 \text{ Kip-ft} \leq M_R = 422.59 \text{ Kip-ft}$$

STATUS OK!

3. Minimum required flexural tension reinforcement in a beam section

$$A_{s,min} = \frac{3 \cdot \sqrt{f_c}}{f_y} \cdot b_w \cdot d = \frac{3 \cdot \sqrt{3000}}{60000} \cdot 11 \cdot 20.00 = 0.60 \text{ in}^2$$

$$A_{s,min} \leq \frac{200}{f_y} \cdot b_w \cdot d = \frac{200}{60000} \cdot 11 \cdot 20.00 = 0.73 \text{ in}^2 \rightarrow A_{s,min} = 0.73 \text{ in}^2$$

4. Maximum required flexural tension reinforcement in a beam section

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

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

$$A_{s,max} = \rho_{max} \cdot b_w \cdot d = 0.01355 \cdot 11 \cdot 20.00 = 2.98 \text{ in}^2$$

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

$$A_{st} = 6.00 \text{ mm}^2 > A_{st,max} = 2.98 \text{ mm}^2$$

STATUS NG!

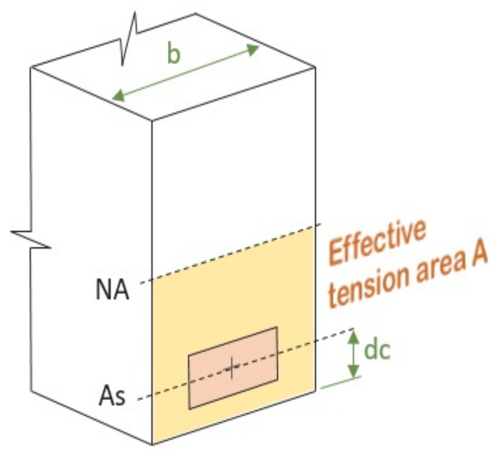
$$A_{st} = 6.00 \text{ mm}^2 \geq A_{st,min} = 0.73 \text{ mm}^2$$

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 = 13.20 in^2
Clear cover c_c = 1.936 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

$$w = 0.076 \cdot \beta_h \cdot f_s \cdot \sqrt[3]{d_c \cdot A} = 0.076 \cdot 1.2 \cdot 36000.00 \cdot \sqrt[3]{2.5 \cdot 13.20} = 0.0105 \text{ in}$$

3. Determine maximum code-permitted bar spacing, s

$$s = 15 \cdot (\frac{40000}{f_s}) - 2.5 \cdot c_c = 15 \cdot (\frac{40000}{36000.00}) - 2.5 \cdot 1.936 = 11.83 \text{ in}$$

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

$$w = 0.0105 \text{ in} \leq w_{lim} = 0.012 \text{ in}$$

STATUS OK!

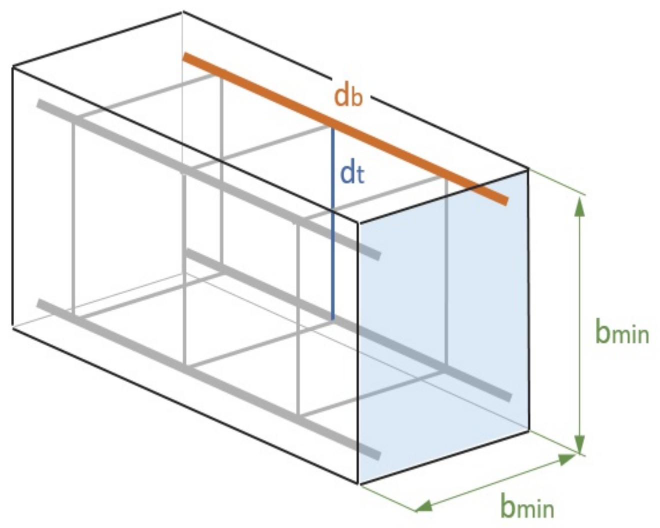
$$c_c = 1.936 \text{ in} \leq s = 11.83 \text{ in}$$

STATUS OK!

CHAPTER 9 (Section 9.7)

Lateral support check for compression steel (Positive bending moment case)

BUCKLING OF SLENDER REINFORCING BARS



Section input data:

Spacing of the stirrups $s = 10$ in

Diameter of the main bars $16 \cdot d_b = 16 \cdot 1.27 = 20.32$ in

Diameter of the transverse reinforcement (stirrups) $48 \cdot d_t = 48 \cdot 0.5 = 24$ in

Smaller dimension of the beam section $b_{min} = 11$ in

Calculate maximum spacing of the stirrups for purpose of buckling bar stability

$$s_{max} = min[16 \cdot d_b, 48 \cdot d_t, b_{min}] = 11 \text{ in}$$

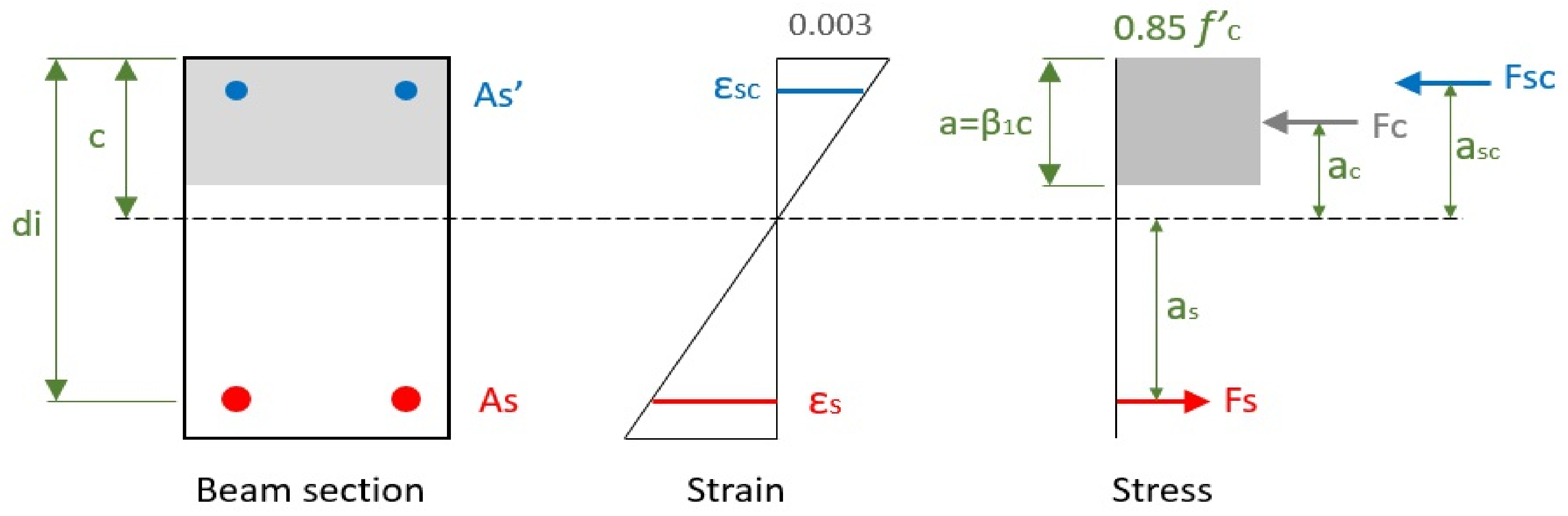
$$s = 10 \text{ in} \leq s_{max} = 11 \text{ in}$$

STATUS OK!

CHAPTER 9 (Section 9.5)

Flexure check (Negative 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 $d_t = 21.1$ in

Given bending moment $M = 0.00$ kip-ft

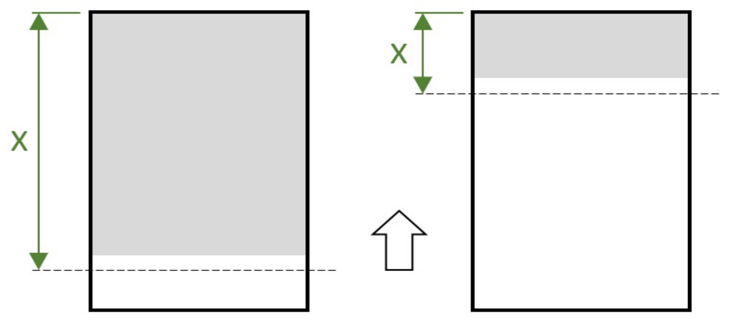
Section Rebar

Depth di (in)	bar diameter (in)	bar area Asi (in^2)
21.10	1.270	1.27
21.10	1.270	1.27
4.70	1.128	1.00
4.70	1.128	1.00
4.70	1.128	1.00
2.50	1.128	1.00
2.50	1.128	1.00
2.50	1.128	1.00

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 $F_c = 0.85 \cdot f_c' \cdot \int_{dA} \cdot \beta_1 \cdot c$
- 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 21.1 to 0 in)

Iter.	c (in)	c/dt	Fc (lbf)	Fcs (lbf)	Fc + Fcs (lbf)	Fs (lbf)	Ratio
1	21.1	1.00	503076.75	360000.00	863076.75	0.00	Infinity
2	20.7	0.98	493015.22	360000.00	853015.22	4509.80	189.147
3	20.3	0.96	482953.68	360000.00	842953.68	9207.50	91.551
4	19.8	0.94	472892.14	360000.00	832892.15	14105.11	59.049
5	19.4	0.92	462830.61	360000.00	822830.61	19215.65	42.821
6	19.0	0.90	452769.07	360000.00	812769.07	24553.33	33.102
7	18.6	0.88	442707.54	360000.00	802707.54	30133.64	26.638
8	18.1	0.86	432646.00	360000.00	792646.00	35973.49	22.034
9	17.7	0.84	422584.47	360000.00	782584.47	42091.43	18.592
10	17.3	0.82	412522.93	360000.00	772522.93	48507.80	15.926
11	16.9	0.80	402461.40	360000.00	762461.40	55245.00	13.801
12	16.5	0.78	392399.86	360000.00	752399.86	62327.69	12.072
13	16.0	0.76	382338.33	360000.00	742338.33	69783.16	10.638
14	15.6	0.74	372276.79	360000.00	732276.79	77641.62	9.431
15	15.2	0.72	362215.26	360000.00	722215.26	85936.67	8.404
16	14.8	0.70	352153.72	357946.51	710100.24	94705.71	7.498
17	14.3	0.68	342092.19	355503.76	697595.95	103990.59	6.708
18	13.9	0.66	332030.65	352912.97	684943.62	113838.18	6.017
19	13.5	0.64	321969.12	350160.25	672129.37	124301.25	5.407
20	13.1	0.62	311907.58	347229.93	659137.52	135439.35	4.867
21	12.7	0.60	301846.05	344104.27	645950.32	147320.00	4.385
22	12.2	0.58	291784.51	340763.03	632547.55	152400.00	4.151
23	11.8	0.56	281722.98	337183.14	618906.12	152400.00	4.061
24	11.4	0.54	271661.44	333338.07	604999.52	152400.00	3.970
25	11.0	0.52	261599.91	329197.23	590797.14	152400.00	3.877
26	10.5	0.50	251538.37	324725.12	576263.49	152400.00	3.781
27	10.1	0.48	241476.84	319880.33	561357.17	152400.00	3.683
28	9.7	0.46	231415.30	314614.26	546029.56	152400.00	3.583
29	9.3	0.44	221353.77	308869.45	530223.22	152400.00	3.479
30	8.9	0.42	211292.23	302577.52	513869.76	152400.00	3.372
31	8.4	0.40	201230.70	295656.40	496887.10	152400.00	3.260
32	8.0	0.38	191169.16	287627.34	478796.50	152400.00	3.142
33	7.6	0.36	181107.63	274606.64	455714.27	152400.00	2.990

34	7.2	0.34	171046.09	260054.08	431100.18	152400.00	2.829
35	6.8	0.32	160984.56	243682.46	404667.02	152400.00	2.655
36	6.3	0.30	150923.02	225127.96	376050.99	152400.00	2.468
37	5.9	0.28	140861.49	203922.82	344784.31	152400.00	2.262
38	5.5	0.26	130799.95	179455.34	310255.30	152400.00	2.036
39	5.1	0.24	120738.42	150909.95	271648.37	152400.00	1.782
40	4.6	0.22	110676.88	120435.59	231112.47	155661.09	1.485
41	4.2	0.20	100615.35	106379.15	206994.50	182087.20	1.137
(Fc + Fcs) < Fs. Updating of iterations							
1	3.8	0.18	90553.81	89199.05	179752.87	214385.78	0.838
2	4.2	0.20	100414.12	106069.29	206483.40	182669.74	1.130
3	4.2	0.20	100212.89	105758.18	205971.07	183254.62	1.124
4	4.2	0.20	100011.66	105445.82	205457.48	183841.85	1.118
5	4.2	0.20	99810.43	105132.20	204942.63	184431.46	1.111
6	4.2	0.20	99609.20	104817.32	204426.52	185023.44	1.105
7	4.2	0.20	99407.97	104501.16	203909.13	185617.82	1.099
8	4.2	0.20	99206.74	104183.72	203390.45	186214.61	1.092
9	4.2	0.20	99005.50	103864.99	202870.49	186813.82	1.086
10	4.1	0.20	98804.27	103544.96	202349.23	187415.48	1.080
11	4.1	0.20	98603.04	103223.62	201826.66	188019.60	1.073
12	4.1	0.20	98401.81	102900.97	201302.78	188626.18	1.067
13	4.1	0.20	98200.58	102576.99	200777.58	189235.25	1.061
14	4.1	0.19	97999.35	102251.69	200251.04	189846.82	1.055
15	4.1	0.19	97798.12	101925.05	199723.17	190460.91	1.049
16	4.1	0.19	97596.89	101597.06	199193.95	191077.53	1.042
17	4.1	0.19	97395.66	101267.71	198663.37	191696.70	1.036
18	4.1	0.19	97194.43	100937.01	198131.43	192318.43	1.030
19	4.1	0.19	96993.20	100604.92	197598.12	192942.74	1.024
20	4.1	0.19	96791.97	100271.46	197063.43	193569.65	1.018
21	4.1	0.19	96590.74	99936.61	196527.35	194199.17	1.012
22	4.0	0.19	96389.51	99600.36	195989.87	194831.32	1.006
23	4.03	0.19	96188.27	99262.71	195450.98	195466.11	1.000

Final value of c is 4.03 in and flexural tension reinforcement area is 5.54 in^2
Working depth of reinforcement $d = 12.22 \text{ in}$

$$e_t = \frac{d_t - c}{c} \cdot (0.003) = \frac{21.1 - 4.03}{4.03} \cdot (0.003) = 0.01269 \geq 0.005 \rightarrow \phi = 0.9$$

2. Calculation moment resistance M_R

$$M_R = \phi \cdot M = \phi \cdot (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) = 0.90 \cdot (223131.21 + 152300.76 + 2629477.88) = 2704.42 \text{ Ibf-in} \\ = 225.37 \text{ Kip-ft}$$

$$M = 0.00 \text{ Kip-ft} \leq M_R = 225.37 \text{ Kip-ft}$$

STATUS OK!

3. Minimum required flexural tension reinforcement in a beam section

$$A_{s,min} = \frac{3 \cdot \sqrt{f_c}}{f_y} \cdot b_w \cdot d = \frac{3 \cdot \sqrt{3000}}{60000} \cdot 11 \cdot 12.22 = 0.37 \text{ in}^2$$

$$A_{s,min} \leq \frac{200}{f_y} \cdot b_w \cdot d = \frac{200}{60000} \cdot 11 \cdot 12.22 = 0.45 \text{ in}^2 \rightarrow A_{s,min} = 0.45 \text{ in}^2$$

4. Maximum required flexural tension reinforcement in a beam section

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

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

$$A_{s,max} = \rho_{max} \cdot b_w \cdot d = 0.01355 \cdot 11 \cdot 12.22 = 1.82 \text{ in}^2$$

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

$$A_{st} = 5.54 \text{ mm}^2 > A_{st,max} = 1.82 \text{ mm}^2$$

STATUS NG!

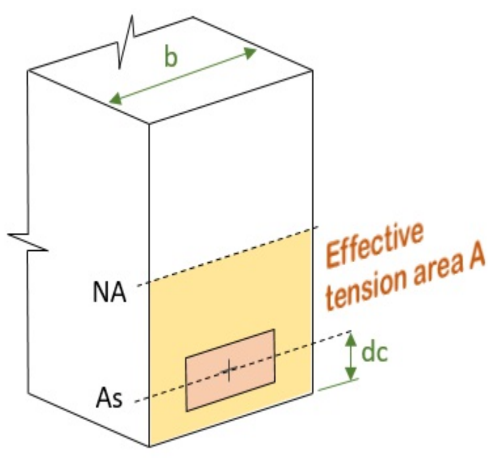
$$A_{st} = 5.54 \text{ mm}^2 \geq A_{st,min} = 0.45 \text{ mm}^2$$

STATUS OK!

CHAPTER 10 (Section 10.6)

Crack check (Negative 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 = 27.50 in^2
Clear cover c_c = 1.865 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

$$w = 0.076 \cdot \beta_h \cdot f_s \cdot \sqrt[3]{d_c \cdot A} = 0.076 \cdot 1.2 \cdot 36000.00 \cdot \sqrt[3]{2.5 \cdot 27.50} = 0.0134 \text{ in}$$

3. Determine maximum code-permitted bar spacing, s

$$s = 15 \cdot (\frac{40000}{f_s}) - 2.5 \cdot c_c = 15 \cdot (\frac{40000}{36000.00}) - 2.5 \cdot 1.865 = 12.00 \text{ in}$$

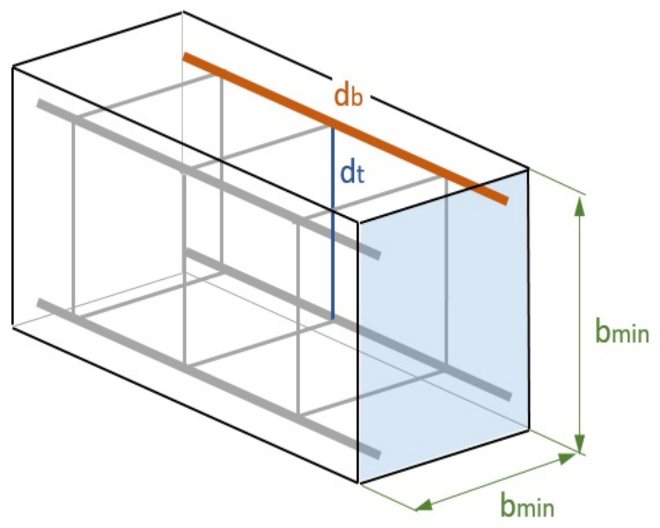
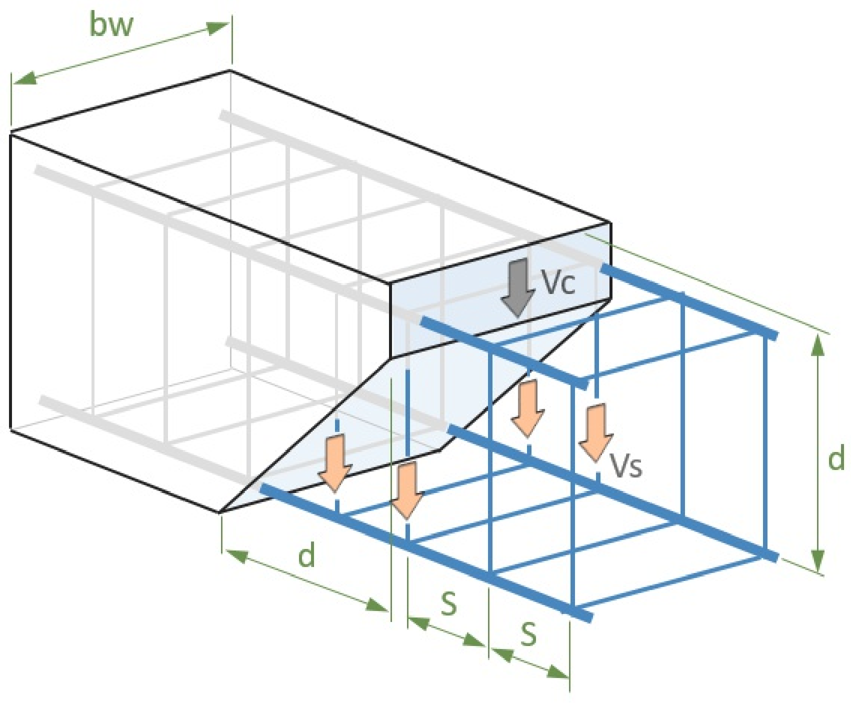
$$s \leq 12 \cdot (\frac{40000}{f_s}) = 12 \cdot (\frac{40000}{36000.00}) = 13.33 \text{ in}$$

$$w = 0.0134 \text{ in} > w_{lim} = 0.012 \text{ in}$$

STATUS NG!

$$c_c = 1.865 \text{ in} \leq s = 12.00 \text{ in}$$

STATUS OK!

CHAPTER 9 (Section 9.7)	**Lateral support check for compression steel (Negative bending moment case)** BUCKLING OF SLENDER REINFORCING BARS  Section input data: Spacing of the stirrups s = 10 in Diameter of the main bars $16 \cdot d_b = 16 \cdot 1.128 = 18.048$ in Diameter of the transverse reinforcement (stirrups) $48 \cdot d_t = 48 \cdot 0.5 = 24$ in Smaller dimension of the beam section $b_{min} = 11$ in Calculate maximum spacing of the stirrups for purpose of buckling bar stability $$s_{max} = min[16 \cdot d_b, 48 \cdot d_t, b_{min}] = 11 \text{ in}$$ $$s = 10 \text{ in} \leq s_{max} = 11 \text{ in}$$	**STATUS OK!**
CHAPTER 11 (Section 11.2)	**Shear check** BEAM ANALYSIS FOR SHEAR (IN PLANE)  Section input data: Working depth of reinforcement d = 20 in Spacing of the stirrups s = 10 in Number of stirrups that are intersected by inclined section n = 3 Web width b_w = 11 in Tensile reinforcement area A_s = 6.00 in^2 Sum of the cross sectional areas of the stirrup legs A_v = 0.39 in^2 Concrete area A_g = 259.60 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 ($A_{v,min}$) $$A_{v,min} = 0.75 \cdot \sqrt{f_c} \cdot \frac{b_w \cdot s}{f_y} = 0.75 \cdot \sqrt{3000} \cdot \frac{11 \cdot 10}{50000} = 0.09 \text{ in}^2$$ $$A_{v,min} = 0.09 \text{ in}^2 < \frac{50 \cdot b_w \cdot s}{f_y} = \frac{50 \cdot 11 \cdot 10}{50000} = 0.11 \text{ in}^2 \rightarrow A_{v,min} = 0.11 \text{ in}^2$$ $$A_v = 0.39 \text{ in}^2 \geq A_{v,min} = 0.11 \text{ in}^2 \rightarrow \text{ area of shear reinforcement is satisfied}$$ 2. Calculate maximum spacing for vertical stirrups (s_{max}) $$S_{max} = \min\left[\frac{d}{2}, 24\right] = 10.00 \text{ in}$$	**STATUS OK!**

	$$s = 10 \text{ in} \leq S_{max} = 10.00 \text{ in} \rightarrow \text{ spacing of stirrups is satisfied}$$ 3. Calculate shear strength of section stirrups (V_s) $$V_s = n \cdot A_v \cdot f_y = 3 \cdot 0.39 \cdot 50000 = 58904.86 \text{ lb} = 58.90 \text{ kip}$$ 4. Calculate shear strength of concrete section (V_c) case beam subjected to shear force only $$V_c = 2 \cdot \lambda \cdot \sqrt{f_c} \cdot b_w \cdot d =$$ $$= 2 \cdot 1 \cdot \sqrt{3000} \cdot 11 \cdot 20 = 24099.79 \text{ lb} = 24.10 \text{ kip}$$ For design check V_c is taken as 24.10 kip 5. Calculate design resisting shear (V_R) $$V_R = \phi \cdot (V_c + V_s) = 0.75 \cdot (24099.79 + 58904.86) = 62.25 \text{ kip}$$ $$V_u = 0.00 \text{ kip} \leq V_R = 62.25 \text{ kip}$$	**STATUS OK!** **STATUS OK!**