REFERENCES	CALCULATIONS	RESULTS

Code: ACI 318-14

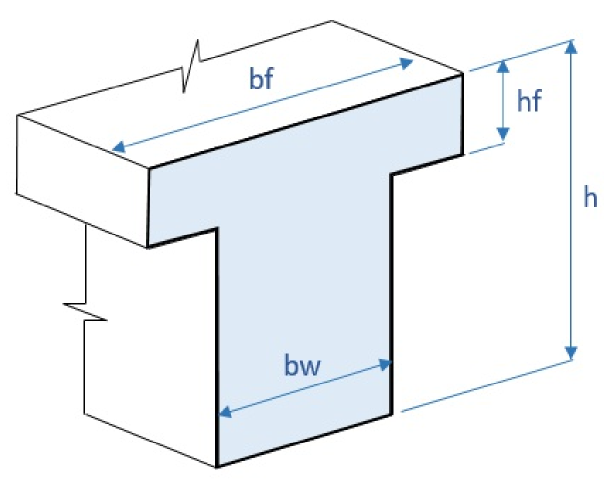
MEMBER #1 (SECTION POSITION 0.0 INCHES) BEAM DESIGN REPORT

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 $h = 14.5\ \mathrm{in}$
Flange width $b_f = 32\ \mathrm{in}$
Flange thickness $h_f = 2\ \mathrm{in}$
Web width $b_w = 10\ \mathrm{in}$
Member length = $240\ \mathrm{in}$

Material properties:
Concrete strength $f_c = 3000\ \mathrm{psi}$
Steel strength of longitudinal rebar $f_y = 60000\ \mathrm{psi}$
Steel strength of shear rebar $f_{yt} = 50000\ \mathrm{psi}$
Permissible crack width $c_w = 0.012\ \mathrm{in}$

Load Combinations (Ultimate Limit State)

For axial force in section:
LC1: USER = $0\ \mathrm{Kip}$

For bending moment in section:
LC1: USER = $0\ \mathrm{Kip\text{-}ft}$

For shear force in section:
LC1: USER = $0\ \mathrm{Kip}$

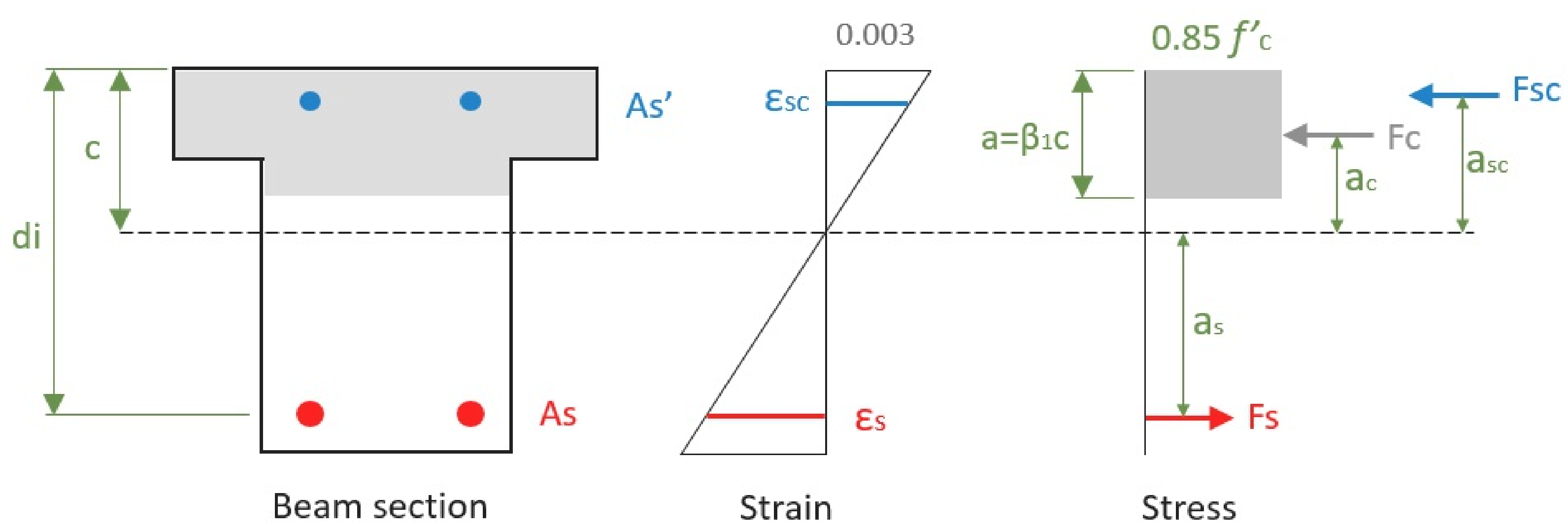
Load Combinations (Serviceability Limit State)

For bending moment in section:
LC1: USER = $0\ \mathrm{Kip\text{-}ft}$

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 $d_t = 12\ \mathrm{in}$
Given bending moment $M = 0.00\ \mathrm{kip\text{-}ft}$

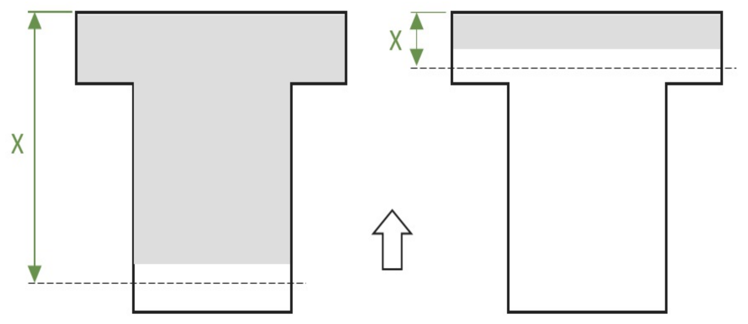
Section Rebar

Depth di (in)	bar diameter (in)	bar area Asi (in^2)
12.00	1.128	1.00
12.00	1.128	1.00
12.00	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 12 to 0 in)

Iter.	c (in)	c/dt	Fc (lbf)	Fcs (lbf)	Fc + Fcs (lbf)	Fs (lbf)	Ratio
1	12.0	1.00	372300.00	0.00	372300.00	0.00	Infinity
2	11.8	0.98	367098.00	0.00	367098.00	5326.53	68.919
3	11.5	0.96	361896.00	0.00	361896.00	10875.00	33.278
4	11.3	0.94	356694.00	0.00	356694.00	16659.57	21.411
5	11.0	0.92	351492.00	0.00	351492.00	22695.65	15.487
6	10.8	0.90	346290.00	0.00	346290.00	29000.00	11.941
7	10.6	0.88	341088.00	0.00	341088.00	35590.91	9.584
8	10.3	0.86	335886.00	0.00	335886.00	42488.37	7.905
9	10.1	0.84	330684.00	0.00	330684.00	49714.29	6.652
10	9.8	0.82	325482.00	0.00	325482.00	57292.68	5.681
11	9.6	0.80	320280.00	0.00	320280.00	65250.00	4.909
12	9.4	0.78	315078.00	0.00	315078.00	73615.38	4.280
13	9.1	0.76	309876.00	0.00	309876.00	82421.05	3.760
14	8.9	0.74	304674.00	0.00	304674.00	91702.70	3.322
15	8.6	0.72	299472.00	0.00	299472.00	101500.00	2.950
16	8.4	0.70	294270.00	0.00	294270.00	111857.14	2.631
17	8.2	0.68	289068.00	0.00	289068.00	122823.53	2.354
18	7.9	0.66	283866.00	0.00	283866.00	134454.55	2.111
19	7.7	0.64	278664.00	0.00	278664.00	146812.50	1.898
20	7.4	0.62	273462.00	0.00	273462.00	159967.74	1.709
21	7.2	0.60	268260.00	0.00	268260.00	174000.00	1.542
22	7.0	0.58	263058.00	0.00	263058.00	180000.00	1.461
23	6.7	0.56	257856.00	0.00	257856.00	180000.00	1.433
24	6.5	0.54	252654.00	0.00	252654.00	180000.00	1.404

25	6.2	0.52	247452.00	0.00	247452.00	180000.00	1.375
26	6.0	0.50	242250.00	0.00	242250.00	180000.00	1.346
27	5.8	0.48	237048.00	0.00	237048.00	180000.00	1.317
28	5.5	0.46	231846.00	0.00	231846.00	180000.00	1.288
29	5.3	0.44	226644.00	0.00	226644.00	180000.00	1.259
30	5.0	0.42	221442.00	0.00	221442.00	180000.00	1.230
31	4.8	0.40	216240.00	0.00	216240.00	180000.00	1.201
32	4.6	0.38	211038.00	0.00	211038.00	180000.00	1.172
33	4.3	0.36	205836.00	0.00	205836.00	180000.00	1.144
34	4.1	0.34	200634.00	0.00	200634.00	180000.00	1.115
35	3.8	0.32	195432.00	0.00	195432.00	180000.00	1.086
36	3.6	0.30	190230.00	0.00	190230.00	180000.00	1.057
37	3.4	0.28	185028.00	0.00	185028.00	180000.00	1.028
(Fc + Fcs) < Fs. Updating of iterations							
1	3.1	0.26	179826.00	0.00	179826.00	180000.00	0.999
2	3.4	0.28	184923.96	0.00	184923.96	180000.00	1.027
3	3.4	0.28	184819.92	0.00	184819.92	180000.00	1.027
4	3.3	0.28	184715.88	0.00	184715.88	180000.00	1.026
5	3.3	0.28	184611.84	0.00	184611.84	180000.00	1.026
6	3.3	0.28	184507.80	0.00	184507.80	180000.00	1.025
7	3.3	0.28	184403.76	0.00	184403.76	180000.00	1.024
8	3.3	0.28	184299.72	0.00	184299.72	180000.00	1.024
9	3.3	0.28	184195.68	0.00	184195.68	180000.00	1.023
10	3.3	0.28	184091.64	0.00	184091.64	180000.00	1.023
11	3.3	0.28	183987.60	0.00	183987.60	180000.00	1.022
12	3.3	0.28	183883.56	0.00	183883.56	180000.00	1.022
13	3.3	0.28	183779.52	0.00	183779.52	180000.00	1.021
14	3.3	0.27	183675.48	0.00	183675.48	180000.00	1.020
15	3.3	0.27	183571.44	0.00	183571.44	180000.00	1.020
16	3.3	0.27	183467.40	0.00	183467.40	180000.00	1.019
17	3.3	0.27	183363.36	0.00	183363.36	180000.00	1.019
18	3.3	0.27	183259.32	0.00	183259.32	180000.00	1.018
19	3.3	0.27	183155.28	0.00	183155.28	180000.00	1.018
20	3.3	0.27	183051.24	0.00	183051.24	180000.00	1.017
21	3.3	0.27	182947.20	0.00	182947.20	180000.00	1.016
22	3.3	0.27	182843.16	0.00	182843.16	180000.00	1.016
23	3.3	0.27	182739.12	0.00	182739.12	180000.00	1.015
24	3.2	0.27	182635.08	0.00	182635.08	180000.00	1.015
25	3.2	0.27	182531.04	0.00	182531.04	180000.00	1.014
26	3.2	0.27	182427.00	0.00	182427.00	180000.00	1.013
27	3.2	0.27	182322.96	0.00	182322.96	180000.00	1.013
28	3.2	0.27	182218.92	0.00	182218.92	180000.00	1.012
29	3.2	0.27	182114.88	0.00	182114.88	180000.00	1.012
30	3.2	0.27	182010.84	0.00	182010.84	180000.00	1.011

31	3.2	0.27	181906.80	0.00	181906.80	180000.00	1.011
32	3.2	0.27	181802.76	0.00	181802.76	180000.00	1.010
33	3.2	0.27	181698.72	0.00	181698.72	180000.00	1.009
34	3.2	0.27	181594.68	0.00	181594.68	180000.00	1.009
35	3.2	0.27	181490.64	0.00	181490.64	180000.00	1.008
36	3.2	0.27	181386.60	0.00	181386.60	180000.00	1.008
37	3.2	0.27	181282.56	0.00	181282.56	180000.00	1.007
38	3.2	0.27	181178.52	0.00	181178.52	180000.00	1.007
39	3.2	0.26	181074.48	0.00	181074.48	180000.00	1.006
40	3.2	0.26	180970.44	0.00	180970.44	180000.00	1.005
41	3.2	0.26	180866.40	0.00	180866.40	180000.00	1.005
42	3.2	0.26	180762.36	0.00	180762.36	180000.00	1.004
43	3.2	0.26	180658.32	0.00	180658.32	180000.00	1.004
44	3.2	0.26	180554.28	0.00	180554.28	180000.00	1.003
45	3.1	0.26	180450.24	0.00	180450.24	180000.00	1.003
46	3.1	0.26	180346.20	0.00	180346.20	180000.00	1.002
47	3.1	0.26	180242.16	0.00	180242.16	180000.00	1.001
48	3.1	0.26	180138.12	0.00	180138.12	180000.00	1.001
49	3.1	0.26	180034.08	0.00	180034.08	180000.00	1.000
50	3.12	0.26	179930.04	0.00	179930.04	180000.00	1.000

Final value of c is 3.12 in and flexural tension reinforcement area is 3.00 in^2
Working depth of reinforcement $d = 12.00 \text{ in}$

$$e_t = \frac{d_t - c}{c} \cdot (0.003) = \frac{12 - 3.12}{3.12} \cdot (0.003) = 0.00852 \geq 0.005 \rightarrow \phi = 0.9$$

2. Calculation moment resistance M_R

$$M_R = \phi \cdot M = \phi \cdot (F_c \cdot a_c + F_{cs} \cdot a_{cs} + F_s \cdot a_s) = 0.90 \cdot (360097.19 + 0.00 + 1597536.00) = 1761.87 \text{ Ibf-in} = 146.82 \text{ Kip-ft}$$

$$M = 0.00 \text{ Kip-ft} \leq M_R = 146.82 \text{ Kip-ft}$$

STATUS OK!

3. Minimum required flexural tension reinforcement in a beam section

$$A_{s,min} = \frac{3 \cdot \sqrt{f_c}}{f_y} \cdot b_w \cdot d = \frac{3 \cdot \sqrt{3000}}{60000} \cdot 10 \cdot 12.00 = 0.33 \text{ in}^2$$

$$A_{s,min} \leq \frac{200}{f_y} \cdot b_w \cdot d = \frac{200}{60000} \cdot 10 \cdot 12.00 = 0.40 \text{ in}^2 \rightarrow A_{s,min} = 0.40 \text{ in}^2$$

4. Maximum required flexural tension reinforcement in a beam section

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

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

$$A_{s,max} = \rho_{max} \cdot b_w \cdot d = 0.01355 \cdot 10 \cdot 12.00 = 1.63 \text{ in}^2$$

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

$$A_{st} = 3.00 \text{ mm}^2 > A_{st,max} = 1.63 \text{ mm}^2$$

STATUS NG!

SkyCiv
CLOUD ENGINEERING SOFTWARE

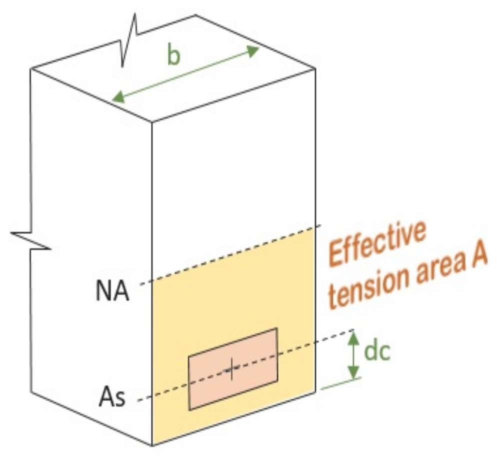
$$A_{st} = 3.00 \text{ mm}^2 \geq A_{st,min} = 0.40 \text{ mm}^2$$

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 = 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

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

3. Determine maximum code-permitted bar spacing, s

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

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

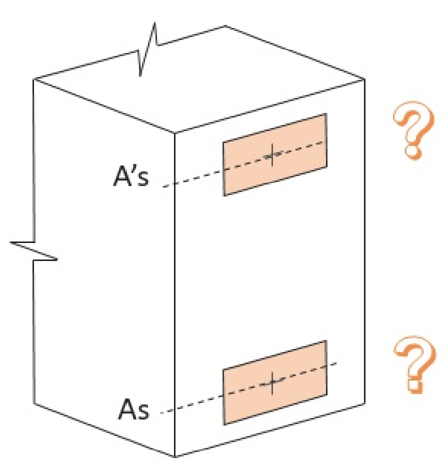
$$w = 0.0114 \text{ in} \leq w_{lim} = 0.012 \text{ in}$$

STATUS OK!

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

STATUS OK!

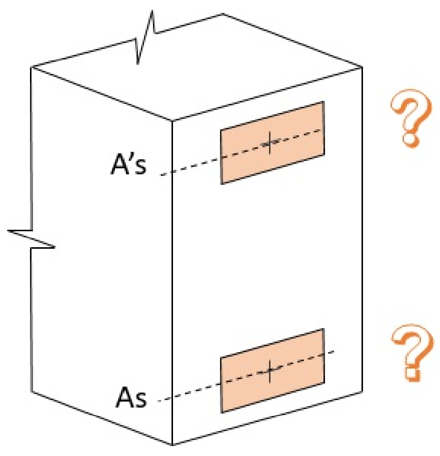
Flexure check (Negative bending moment case)



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)



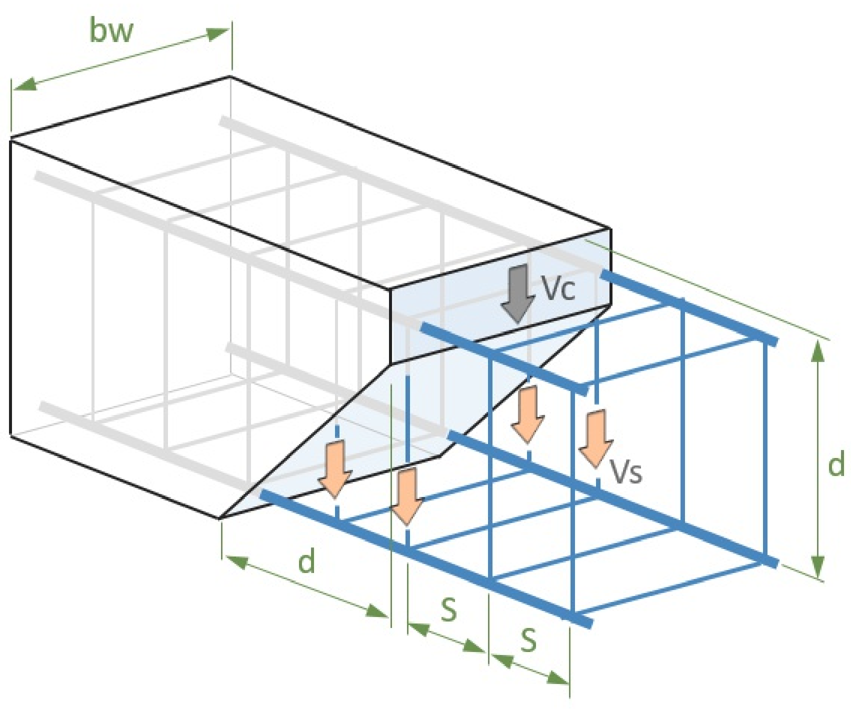
Bottom Reinforcement is absent in the section. Design checks can't be performed. But as acting moment value is equal to zero no need to check.

STATUS OK!

CHAPTER 11 (Section 11.2)

Shear check

BEAM ANALYSIS FOR SHEAR (IN PLANE)



Section input data:
Working depth of reinforcement d = 12 in
Spacing of the stirrups s = 10 in
Number of stirrups that are intersected by inclined section n = 2
Web width b_w = 10 in
Tensile reinforcement area A_s = 3.00 in^2
Sum of the cross sectional areas of the stirrup legs A_v = 0.39 in^2
Concrete area A_g = 189.00 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 ($A_{v,min}$)

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

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

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

STATUS OK!

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

$$S_{max} = \min\left[\frac{d}{2}, 24\right] = 6.00\,\text{in}$$

$$s = 10\,\text{in} > S_{max} = 6.00\,\text{in} \rightarrow \text{ spacing of stirrups is not satisfied}$$

STATUS NG!

3. Calculate shear strength of section stirrups (V_s)

$$V_s = n \cdot A_v \cdot f_y = 2 \cdot 0.39 \cdot 50000 = 39269.91\,\text{lb} = 39.27\,\text{kip}$$

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

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

$$= 2 \cdot 1 \cdot \sqrt{3000} \cdot 10 \cdot 12 = 13145.34\,\text{lb} = 13.15\,\text{kip}$$

	For design check V_c is taken as $13.15\ \mathrm{kip}$ 5. Calculate design resisting shear (V_R) $$V_R = \phi \cdot (V_c + V_s) = 0.75 \cdot (13145.34 + 39269.91) = 39.31\ \mathrm{kip}$$ $$V_u = 0.00\ \mathrm{kip} \leq V_R = 39.31\ \mathrm{kip}$$	**STATUS OK!**