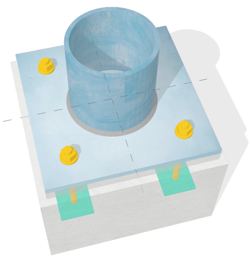
REFERENCES	CALCULATIONS	RESULTS

SkyCiv Base Plate and Anchor Rod Design

Shear and Compression Design Example for AISC

Project Information:
Design Code: AISC 360-22/360-16 (LRFD) and ACI 318-19

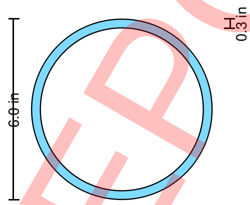


Load Combinations:

Load Combination	Axial Load, Nx (kip)	Strong-axis Shear, Vy (kip)	Weak-axis Shear, Vz (kip)
1	25.00	3.00	1.00

Note(s):
1. Positive Nx (+) indicates compression; negative Nx (-) indicates tension.
2. Shear forces are assumed to be distributed to all anchors in the base plate.
3. Engineering methods other than welded plate washers are assumed to be used to distribute shear forces.

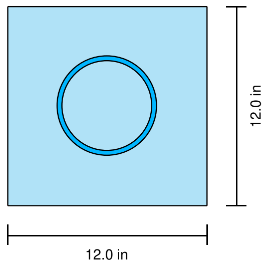
Column Properties:



Symbol	Description	Value
$Section$	Column section	HSS6x0.312
d_{col}	Column outside diameter	6.000 in
t_{col}	Column thickness	0.291 in
A_{col}	Column area	5.220 in^2
F_{y_col}	Column yield stress (A36)	36.000 ksi
F_{u_col}	Column tensile stress (A36)	58.000 ksi

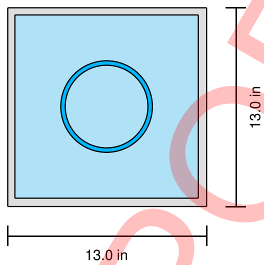
REFERENCES	CALCULATIONS	RESULTS

Base Plate Properties:



Symbol	Description	Value
B_{bp}	Base plate width	12.000 in
L_{bp}	Base plate length	12.000 in
t_{bp}	Base plate thickness	0.500 in
A_{bp}	Base plate plan area	144.000 in^2
F_{y_bp}	Base plate yield stress (A36)	36.000 ksi
F_{u_bp}	Base plate tensile stress (A36)	58.000 ksi
t_{grout}	Grout thickness	0.750 in

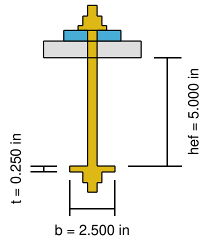
Concrete Properties:



Symbol	Description	Value
B_{conc}	Concrete width	13.000 in
L_{conc}	Concrete length	13.000 in
t_{conc}	Concrete thickness	8.000 in
A_{conc}	Concrete plan area	169.000 in^2
f_c'	Concrete compressive strength (3000)	3.000 ksi
λ	Factor for normal-weight concrete	1.000
—	Concrete assumption: cracked or uncracked	Cracked

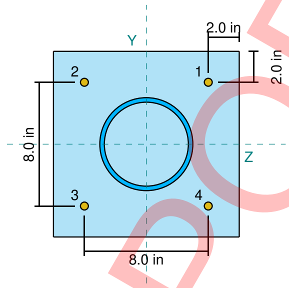
REFERENCES	CALCULATIONS	RESULTS

Anchor Information:



Symbol	Description	Value
d_a	Anchor rod diameter	0.500 in
h_{ef}	Anchor rod effective embedment length	5.000 in
b_{embed_plate}	Anchor embed plate side dimension	2.500 in
t_{embed_plate}	Anchor embed plate thickness	0.250 in
F_{y_anc}	Anchor rod yield stress (A325)	92.000 ksi
F_{u_anc}	Anchor rod tensile stress (A325)	120.000 ksi
F_{y_ep}	Embed plate yield stress (A36)	36.000 ksi
t_{pw}	Plate washer thickness	0.000 in

Anchor Layout:



Symbol	Description	Value
n_a	Total number of anchor rods	4.000
s_z	Spacing of anchor rods along Z-axis	8.000 in
s_y	Spacing of anchor rods along Y-axis	8.000 in
$l_{edge,z}$	Base plate edge distance along Z-axis	2.000 in
$l_{edge,y}$	Base plate edge distance along Y-axis	2.000 in

REFERENCES	CALCULATIONS	RESULTS

Anchor Data Summary:

ID	Z (in)	Y (in)
1	4.000	4.000
2	-4.000	4.000
3	-4.000	-4.000
4	4.000	-4.000

ID	Anchor in Tension Zone	Breakout (Tension)	Side-Face Blowout (Y-direction)	Side-Face Blowout (Z-direction)
1	✓	BO Group 1	N/A	N/A
2	✓	BO Group 1	N/A	N/A
3	✓	BO Group 1	N/A	N/A
4	✓	BO Group 1	N/A	N/A

ID	Shear Direction Resisted	Breakout (Vy Shear)	Breakout (Vz Shear)	Pryout (Vy Shear)	Pryout (Vz Shear)
1	All Load Directions	Single	Single	PO Group 1	PO Group 1
2	All Load Directions	Single	Single	PO Group 1	PO Group 1
3	All Load Directions	Single	Single	PO Group 1	PO Group 1
4	All Load Directions	Single	Single	PO Group 1	PO Group 1

Weld Properties:

Symbol	Description	Value
$Type$	Weld Type	Fillet
W	Fillet weld size	0.250 in
E_w	Weld throat	0.177 in
F_{exx}	Filler metal classification strength (E70xx)	70.000 ksi

REFERENCES	CALCULATIONS	RESULTS

Summary of Detailing Checks

Check Name	Dimensions (Min/Max/Actual)	Status	Reference
Weld Size Requirement	Min: 0.19 in, Max: 0.75 in, Actual: 0.25 in	PASS	AISC 360-22 Table J2.4
Anchor Clearance	Min: 1.19 in in, Actual: 2.42 in	PASS	SkyCiv Recommendation
Base Plate Edge Distance	Min: 0.81 in in, Actual: 2.00 in	PASS	AISC 360-22 Table J3.4 & Table J3.5
Minimum Number of Anchors	Min: 4.00, Actual: 4.00	PASS	AISC Design Guide 1 3rd Ed. Section 4.2
Embedded Plate Dimension	Min: 2.05 in in, Actual: 2.50 in	PASS	SkyCiv Recommendation
Min. Anchor Spacing to Prevent Concrete Splitting	Min: 2.05 in in, Actual: 8.00 in	PASS	ACI 318-19 Table 17.9.2(a)
Min. Concrete Cover to Prevent Concrete Splitting	Min: 1.50 in in, Actual: 2.50 in	PASS	ACI 318-19 Table 20.5.1.3.1

✔ All Detailing Checks Passed

The design geometry meets requirements.

Summary of Design Checks

Design Check	Governing LC	Demand	Capacity	DCR	Result
Weld Capacity	1	1.34	5.58	0.24	PASS
Column Bearing Capacity	1	25.00	253.69	0.10	PASS
Base Plate Yield Capacity (Compression Load)	1	1.12	2.02	0.56	PASS
Concrete Bearing Capacity	1	25.00	258.57	0.10	PASS
Concrete Breakout Capacity (Vy Shear)	1	0.75	0.79	0.95	PASS
Concrete Breakout Capacity (Vz Shear)	1	0.25	0.79	0.32	PASS
Concrete Pryout Capacity	1	3.16	17.44	0.18	PASS
Anchor Rod Shear Capacity	1	0.79	3.98	0.20	PASS

✔ All Design Checks Passed

The design is adequate.

REFERENCES	CALCULATIONS	RESULTS

Load Combination No. 1: Compression Load and Shear Load

Check No. 1: Weld Capacity

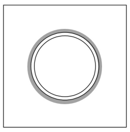


Fig. Welds on column to base plate.

Calculate total weld length around Pipe or Round HSS

$d_{col} = 6\text{ in}$ - Column outside diameter

L_{weld} - Total length of weld

$$L_{weld} = \pi d_{col} = \pi \times 6\text{ in} = 18.85\text{ in}$$

Calculate load per unit length of weld

Compute the resultant stress from compression and shear loads.

$N_x = 25\text{ kip}$ - Compression load

$V_y = 3\text{ kip}$ - Vy shear load

$V_z = 1\text{ kip}$ - Vz shear load

c_u - Required weld strength per unit length

$$c_u = \frac{N_x}{L_{weld}} = \frac{25\text{ kip}}{18.85\text{ in}} = 1.3263\text{ kip/in}$$

v_{uy} - Required weld strength per unit length

$$v_{uy} = \frac{V_y}{L_{weld}} = \frac{3\text{ kip}}{18.85\text{ in}} = 0.15915\text{ kip/in}$$

v_{uz} - Required weld strength per unit length

$$v_{uz} = \frac{V_z}{L_{weld}} = \frac{1\text{ kip}}{18.85\text{ in}} = 0.053052\text{ kip/in}$$

r_u - Required weld strength per unit length

$$r_u = \sqrt{\left((c_u)^2\right) + \left((v_{uy})^2\right) + \left((v_{uz})^2\right)}$$

$$r_u = \sqrt{\left((1.3263\text{ kip/in})^2\right) + \left((0.15915\text{ kip/in})^2\right) + \left((0.053052\text{ kip/in})^2\right)}$$

$$r_u = 1.3369\text{ kip/in}$$

Calculate fillet weld capacity

$F_{exx} = 70\text{ ksi}$ - Filler metal classification strength (E70xx)

$E_w = 0.177\text{ in}$ - Weld throat

$\theta = 0$ - Angle of load in radians

$\phi = 0.75$ - Weld resistance factor

AISC 360-22 Chapter J2.4 Eq. (J2-5)

k_{ds} - Directional strength increase factor

$$k_{ds} = 1.0 + 0.5(\sin(\theta))^{1.5} = 1 + 0.5 \times (\sin(0))^{1.5} = 1$$

AISC 360-22 Chapter J2.4 Eq. (J2-4)

ϕr_n - Design strength of fillet welds per unit length

$$\phi r_n = \phi 0.6 F_{exx} E_w k_{ds} = 0.75 \times 0.6 \times 70\text{ ksi} \times 0.177\text{ in} \times 1 = 5.5755\text{ kip/in}$$

Calculate base metal capacities

$F_{u\_col} = 58\text{ ksi}$ - Column tensile stress (A36)

$t_{col} = 0.291\text{ in}$ - Column thickness

$F_{u\_bp} = 58\text{ ksi}$ - Base plate tensile stress (A36)

$t_{bp} = 0.5\text{ in}$ - Base plate thickness

$\phi = 0.75$ - Shear rupture resistance factor

AISC 360-22 Eq. J4-4

$\phi r_{nbm,col}$ - Design shear rupture strength of base metal (column)

$$\phi r_{nbm,col} = \phi 0.6 F_{u\_col} t_{col} = 0.75 \times 0.6 \times 58\text{ ksi} \times 0.291\text{ in} = 7.5951\text{ kip/in}$$

AISC 360-22 Eq. J4-4

$\phi r_{nbm,bp}$ - Design shear rupture strength of base metal (base plate)

$$\phi r_{nbm,bp} = \phi 0.6 F_{u\_bp} t_{bp} = 0.75 \times 0.6 \times 58\text{ ksi} \times 0.5\text{ in} = 13.05\text{ kip/in}$$

AISC 360-22 Eq. J4-4

ϕr_{nbm} - Governing design shear rupture strength of base metal

$$\phi r_{nbm} = \min(\phi r_{nbm,bp},\ \phi r_{nbm,col}) = \min(13.05\text{ kip/in},\ 7.5951\text{ kip/in}) = 7.5951\text{ kip/in}$$

Result:

PASS = 0.24

DCR - Demand over capacity ratio, comparing two conditions:

$$DCR = \max\left(\frac{r_u}{\phi r_{nbm}}, \frac{r_u}{\phi r_n}\right)$$

$$DCR = \max\left(\frac{1.3369\ \text{kip/in}}{7.5951\ \text{kip/in}}, \frac{1.3369\ \text{kip/in}}{5.5755\ \text{kip/in}}\right) = 0.23977$$

Go back to summary results table.

Check No. 2: Column Bearing Capacity

Calculate bearing load on column member

$N_x = 25\ \text{kip}$ - Compression load

R_u - Required column bearing strength

$$R_u = N_x = 25\ \text{kip}$$

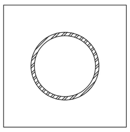


Fig. Bearing area of column.

Calculate bearing capacity of column member

$F_{y\_col} = 36\ \text{ksi}$ - Column yield stress (A36)

$A_{col} = 5.22\ \text{in}^2$ - Column area

$\phi = 0.75$ - Steel bearing resistance factor

AISC 360-22 Chapter J7 Eq. (J7-1), Finished bearing plate per AISC 360-22 Chapter M4.4

ϕR_n - Design bearing strength

$$\phi R_n = \phi 1.8 F_{y\_col} A_{col} = 0.75 \times 1.8 \times 36\ \text{ksi} \times 5.22\ \text{in}^2 = 253.69\ \text{kip}$$

Result:

PASS = 0.10

DCR - Demand-to-Capacity Ratio

$$DCR = \frac{R_u}{\phi R_n} = \frac{25\ \text{kip}}{253.69\ \text{kip}} = 0.098545$$

Go back to summary results table.

Check No. 3: Base Plate Yield Capacity (Compression Load)

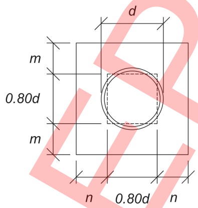


Fig. Cantilever lengths of base plate.

Calculate critical cantilever length for Round HSS or Pipe column

$d_{col} = 6\ \text{in}$ - Column outside diameter

$L_{bp} = 12\ \text{in}$ - Base plate length

$B_{bp} = 12\ \text{in}$ - Base plate width

AISC Design Guide 01 3rd Ed. Section 4.3.1

l - Critical cantilever length of base plate

$$l = \max\left(\frac{\mathrm{L_{bp}} - 0.8\mathrm{d_{col}}}{2}, \frac{\mathrm{B_{bp}} - 0.8\mathrm{d_{col}}}{2}\right)$$

$$l = \max\left(\frac{12\ \text{in} - 0.8 \times 6\ \text{in}}{2}, \frac{12\ \text{in} - 0.8 \times 6\ \text{in}}{2}\right) = 3.6\ \text{in}$$

Calculate applied moment load per unit length on critical section of base plate due to compression from concrete

$N_x = 25\ \text{kip}$ - Compression load

$B_{bp} = 12\ \text{in}$ - Base plate width

$L_{bp} = 12\ \text{in}$ - Base plate length

AISC Design Guide 01 3rd Ed. Section 4.3.1

$l = 3.6\ \text{in}$

AISC Design Guide 01 2nd Ed.

m_u - Required flexural yielding strength of base plate

$$m_u = \left(\frac{N_x}{B_{bp} L_{bp}}\right)\left(\frac{l^2}{2}\right)$$

$$m_u = \left(\frac{25\ \text{kip}}{12\ \text{in} \times 12\ \text{in}}\right) \times \left(\frac{3.6\ \text{in}^2}{2}\right) = 1.125\ \text{kip·in/in}$$

Calculate flexural yielding capacity of base plate per unit length due to compression from concrete

$F_{y\_bp} = 36\ \text{ksi}$ - Base plate yield stress (A36)

$t_{bp} = 0.5\ \text{in}$ - Base plate thickness

$\phi = 0.9$ - Flexural resistance factor

AISC Manual 15th Ed. Part 14 pg.14-6, AISC 360-22 Chapter F2 Eq. (F2-1)

ϕm_n - Design flexural yielding strength of base plate per unit length

$$\phi m_n = \phi F_{y\_bp} \left(\frac{(t_{bp})^2}{4} \right) = 0.9 \times 36 \text{ ksi} \times \left(\frac{(0.5 \text{ in})^2}{4} \right) = 2.025 \text{ kip·in/in}$$

Result:

PASS = 0.56

DCR - Demand-to-Capacity Ratio

$$DCR = \frac{m_u}{\phi m_n} = \frac{1.125 \text{ kip·in/in}}{2.025 \text{ kip·in/in}} = 0.55556$$

[Go back to summary results table.]

Check No. 4: Concrete Bearing Capacity

Calculate bearing load on concrete

$N_x = 25 \text{ kip}$ - Compression load

P_u - Required concrete bearing strength

$$P_u = N_x = 25 \text{ kip}$$

The restraining side is equal along the concrete width and concrete length.

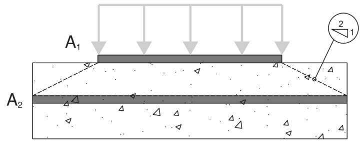


Fig. Bearing area on base plate (A1) and maximum support area on concrete (A2).

Calculate bearing area on base plate

$L_{bp} = 12 \text{ in}$ - Base plate length

$B_{bp} = 12 \text{ in}$ - Base plate width

AISC 360-22 Chapter J8, ACI 318-19 Sect. 22.8.3.2 and Fig. R22.8.3.2

A_1 - Area of bearing on base plate (assume full area)

$$A_1 = L_{bp} B_{bp} = 12 \text{ in} \times 12 \text{ in} = 144 \text{ in}^2$$

Calculate maximum bearing area on concrete foundation

$N_{A2} = 13 \text{ in}$ - Length of max bearing area on support (A2)

$B_{A2} = 13 \text{ in}$ - Width of max bearing area on support (A2)

AISC 360-22 Chapter J8, ACI 318-19 Sect. 22.8.3.2 and Fig. R22.8.3.2

A_2 - Maximum area of bearing on concrete (assume 1:2 slope)

$$A_2 = N_{A2} B_{A2} = 13 \text{ in} \times 13 \text{ in} = 169 \text{ in}^2$$

Calculate bearing capacity of concrete

$f_c^{'} = 3 \text{ ksi}$ - Concrete compressive strength (3000)

AISC 360-22 Chapter J8, ACI 318-19 Sect. 22.8.3.2 and Fig. R22.8.3.2

$A_1 = 144 \text{ in}^2$

AISC 360-22 Chapter J8, ACI 318-19 Sect. 22.8.3.2 and Fig. R22.8.3.2

$A_2 = 169 \text{ in}^2$

$\phi = 0.65$ - Concrete bearing resistance factor

AISC 360-22 Chapter J8 Eq. (J8-2)

ϕP_p - Design bearing strength of concrete crushing

$$\phi P_p = \phi \left(\min \left(0.85 \left(f_c^{'} \right) A_1 \sqrt{\frac{A_2}{A_1}},\ 1.7 \left(f_c^{'} \right) A_1 \right) \right)$$

$$\phi P_p = 0.65 \times \left(\min \left(0.85 \times (3 \text{ ksi}) \times 144 \text{ in}^2 \times \sqrt{\frac{169 \text{ in}^2}{144 \text{ in}^2}},\ 1.7 \times (3 \text{ ksi}) \times 144 \text{ in}^2 \right) \right)$$

$$\phi P_p = 258.57 \text{ kip}$$

Result:

PASS = 0.10

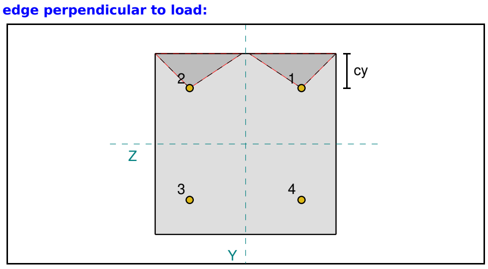
DCR - Demand-to-Capacity Ratio

$$DCR = \frac{P_u}{\phi P_p} = \frac{25 \text{ kip}}{258.57 \text{ kip}} = 0.096686$$

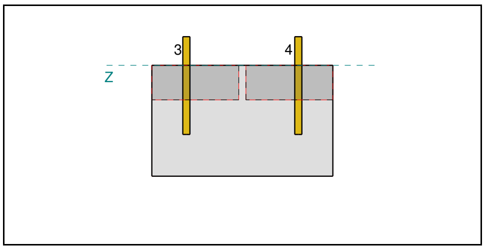
[Go back to summary results table.]

Check No. 5: Concrete Breakout Capacity (Vy Shear)

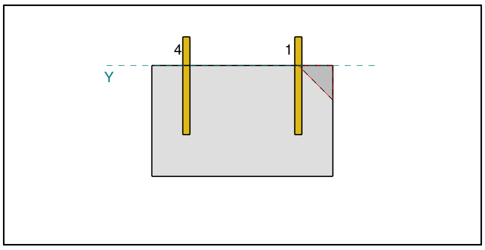
For breakout capacity on failure edge perpendicular to load:



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate shear load per single anchor

$V_y = 3\ \text{kip}$ - Vy shear load

$n_a = 4$ - Total number of anchor rods

$V_{fa\perp}$ - Required concrete breakout shear strength of a single anchor with failure edge perpendicular to load

$$V_{fa\perp} = \frac{V_y}{n_a} = \frac{3\ \text{kip}}{4} = 0.75\ \text{kip}$$

Determine if the support is a narrow concrete member

ACI 318-19 Clause 17.7.2.1.2

The support is not a narrow member.

Symbol	Description	Value
$c_{left,s1}$	Single anchor concrete edge distance (left)	10.500 in
$c_{right,s1}$	Single anchor concrete edge distance (right)	2.500 in
$c_{top,s1}$	Single anchor concrete edge distance (top)	2.500 in
$c_{bottom,s1}$	Single anchor concrete edge distance (bottom)	10.500 in
t_{conc}	Concrete thickness	8.000 in
$c_{a1,s1}$	Single anchor distance to failure edge (+Vy shear)	2.500 in
	Narrow Member	FALSE

Calculation for critical anchor: Anchor ID 1.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.7.2.1.2

$c_{a1,s1} = 2.5\ \text{in}$ - Single anchor distance to failure edge (+Vy shear)

ACI 318-19 Eq. 17.7.2.1.3

A_{Vco} - Maximum projected area for a single anchor

$$A_{Vco} = 4.5(c_{a1,s1})^2 = 4.5 \times (2.5\ \text{in})^2 = 28.125\ \text{in}^2$$

Calculate width of actual projected area on failure surface

ACI 318-19 Clause 17.7.2.1.1

B_{Vc} - Actual length of concrete cone for a single anchor

$$B_{Vc} = \min\left(\text{c}_{\text{left,s1}},\ 1.5\text{c}_{\text{a1,s1}}\right) + \min\left(\text{c}_{\text{right,s1}},\ 1.5\text{c}_{\text{a1,s1}}\right)$$

$$B_{Vc} = \min\left(10.5\ \text{in},\ 1.5 \times 2.5\ \text{in}\right) + \min\left(2.5\ \text{in},\ 1.5 \times 2.5\ \text{in}\right) = 6.25\ \text{in}$$

Calculate height of actual projected area on failure surface

ACI 318-19 Clause 17.7.2.1.1

H_{Vc} - Actual height of projected area

$$H_{Vc} = \min\left(1.5\text{c}_{\text{a1,s1}},\ \text{t}_{\text{conc}}\right) = \min\left(1.5 \times 2.5\ \text{in},\ 8\ \text{in}\right) = 3.75\ \text{in}$$

Calculate actual projected area

ACI 318-19 Clause 17.7.2.1.1

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc} H_{Vc} = 6.25\ \text{in} \times 3.75\ \text{in} = 23.438\ \text{in}^2$$

Calculate modification factor for lightweight concrete

$\lambda = 1$ - Factor for normal-weight concrete

ACI 318-19 Table 17.2.4.1

λ_a - Modification factor for lightweight concrete

$$\lambda_a = 1.0\lambda = 1 \times 1 = 1$$

Calculate load bearing length of the anchor

$h_{ef} = 5\ \text{in}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 5\ \text{in}$$

Calculate basic single anchor breakout strength

$f_c^{'} = 3\ \text{ksi}$ - Concrete compressive strength (3000)

$d_a = 0.5\ \text{in}$ - Anchor rod diameter

ACI 318-19 Eq. 17.7.2.2.1a

V_{b1} - Basic single anchor breakout strength condition 1

$$V_{b1} = 7\left(\frac{\min(\mathrm{l_e},\ 8\mathrm{d_a})}{d_a}\right)^{0.2}\sqrt{\frac{d_a}{in}}\lambda_a\sqrt{\frac{f_c^{'}}{psi}}\left(\frac{c_{a1,s1}}{in}\right)^{1.5} lbf$$

$$V_{b1} = 7\times\left(\frac{\min(5\ \text{in},\ 8\times0.5\ \text{in})}{0.5\ \text{in}}\right)^{0.2}\times\sqrt{\frac{0.5\ \text{in}}{1\ \text{in}}}\times1\times\sqrt{\frac{3\ \text{ksi}}{0.001\ \text{ksi}}}\times\left(\frac{2.5\ \text{in}}{1\ \text{in}}\right)^{1.5}\times0.001\ \text{kip}$$

$$V_{b1} = 1.6243\ \text{kip}$$

ACI 318-19 Eq. 17.7.2.2.1b

V_{b2} - Basic single anchor breakout strength condition 2

$$V_{b2} = 9\lambda_a\sqrt{\frac{f_c^{'}}{psi}}\left(\frac{c_{a1,s1}}{in}\right)^{1.5} lbf$$

$$V_{b2} = 9\times1\times\sqrt{\frac{3\ \text{ksi}}{0.001\ \text{ksi}}}\times\left(\frac{2.5\ \text{in}}{1\ \text{in}}\right)^{1.5}\times0.001\ \text{kip} = 1.9486\ \text{kip}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min(\mathrm{V_{b1}},\ \mathrm{V_{b2}}) = \min(1.6243\ \text{kip},\ 1.9486\ \text{kip}) = 1.6243\ \text{kip}$$

Calculate edge effect factor

$c_{a2,s1} = 2.5\ \text{in}$ - Single anchor distance to parallel edge (+Vy shear)

ACI 318-19 Clause 17.7.2.4.1a and 17.7.2.4.1b

$\Psi_{ed,V}$ - Breakout edge effect factor

$$\Psi_{ed,V} = \min\left(1.0,\ 0.7+0.3\left(\frac{\mathrm{c_{a2,s1}}}{1.5\mathrm{c_{a1,s1}}}\right)\right) = \min\left(1,\ 0.7+0.3\times\left(\frac{2.5\ \text{in}}{1.5\times2.5\ \text{in}}\right)\right) = 0.9$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1

$\Psi_{h,V}$ - Breakout thickness factor

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5\mathrm{c_{a1,s1}}}{\mathrm{t_{conc}}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5\times2.5\ \text{in}}{8\ \text{in}}},\ 1\right) = 1$$

Calculate concrete breakout capacity at the perpendicular edge

$\phi = 0.65$ - Concrete shear resistance factor

ACI 318-19 Clause 17.7.2.5.1

$\Psi_{c,V} = 1$ - Breakout cracking factor (shear)

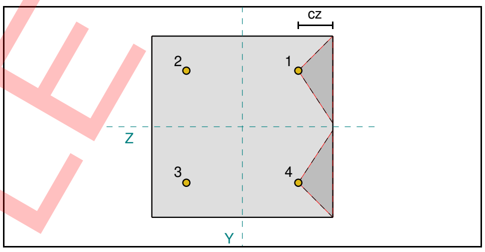
ACI 318-19 Clause 17.7.2.1(a)

$\phi V_{cb\perp}$ - Design concrete breakout strength in shear of a single anchor (perpendicular edge)

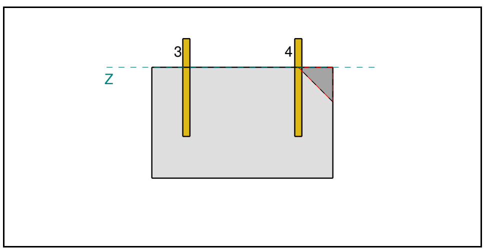
$$\phi V_{cb\perp} = \phi\left(\frac{A_{Vc}}{A_{Vco}}\right)\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$$

$$\phi V_{cb\perp} = 0.65\times\left(\frac{23.438\ \text{in}^2}{28.125\ \text{in}^2}\right)\times0.9\times1\times1\times1.6243\ \text{kip} = 0.79186\ \text{kip}$$

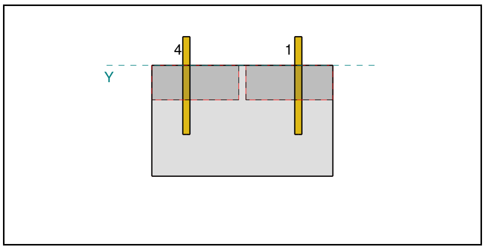
For breakout capacity on failure edge parallel to load:



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate shear load per single anchor

$V_y = 3\ \mathrm{kip}$ - Vy shear load

$n_a = 4$ - Total number of anchor rods

Determine if the support is a narrow concrete member

ACI 318-19 Clause 17.7.2.1.2

The support is not a narrow member.

Symbol	Description	Value
$c_{left,s1}$	Single anchor concrete edge distance (left)	10.500 in
$c_{right,s1}$	Single anchor concrete edge distance (right)	2.500 in
$c_{top,s1}$	Single anchor concrete edge distance (top)	2.500 in
$c_{bottom,s1}$	Single anchor concrete edge distance (bottom)	10.500 in
t_{conc}	Concrete thickness	8.000 in
$c_{a1,s1}$	Single anchor distance to failure edge (+Vz shear)	2.500 in
	Narrow Member	FALSE

Calculation for critical anchor: Anchor ID 1.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.7.2.1.2

$c_{a1,s1} = 2.5\ \mathrm{in}$ - Single anchor distance to failure edge (+Vz shear)

ACI 318-19 Eq. 17.7.2.1.3

A_{Vco} - Maximum projected area for a single anchor

$$A_{Vco} = 4.5(c_{a1,s1})^2 = 4.5 \times (2.5\ \mathrm{in})^2 = 28.125\ \mathrm{in}^2$$

Calculate width of actual projected area on failure surface

ACI 318-19 Clause 17.7.2.1.1

B_{Vc} - Actual length of concrete cone for a single anchor

$$B_{Vc} = \min\left(\mathrm{c_{top,s1}},\ 1.5\mathrm{c_{a1,s1}}\right) + \min\left(\mathrm{c_{bottom,s1}},\ 1.5\mathrm{c_{a1,s1}}\right)$$

$$B_{Vc} = \min\left(2.5\ \mathrm{in},\ 1.5 \times 2.5\ \mathrm{in}\right) + \min\left(10.5\ \mathrm{in},\ 1.5 \times 2.5\ \mathrm{in}\right) = 6.25\ \mathrm{in}$$

Calculate height of actual projected area on failure surface

ACI 318-19 Clause 17.7.2.1.1

H_{Vc} - Actual height of projected area

$$H_{Vc} = \min\left(1.5\mathrm{c_{a1,s1}},\ \mathrm{t_{conc}}\right) = \min\left(1.5 \times 2.5\ \mathrm{in},\ 8\ \mathrm{in}\right) = 3.75\ \mathrm{in}$$

Calculate actual projected area

ACI 318-19 Clause 17.7.2.1.1

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc}H_{Vc} = 6.25\ \mathrm{in} \times 3.75\ \mathrm{in} = 23.438\ \mathrm{in}^2$$

Calculate modification factor for lightweight concrete

$\lambda = 1$ - Factor for normal-weight concrete

ACI 318-19 Table 17.2.4.1

λ_a - Modification factor for lightweight concrete

$$\lambda_a = 1.0\lambda = 1 \times 1 = 1$$

Calculate load bearing length of the anchor

$h_{ef} = 5\ \mathrm{in}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 5\ \mathrm{in}$$

Calculate basic single anchor breakout strength

$f_c^{'} = 3\ \mathrm{ksi}$ - Concrete compressive strength (3000)

$d_a = 0.5\ \mathrm{in}$ - Anchor rod diameter

ACI 318-19 Eq. 17.7.2.2.1a

V_{b1} - Basic single anchor breakout strength condition 1

$$V_{b1} = 7\left(\frac{\min\left(\mathrm{l_e},\ 8\mathrm{d_a}\right)}{d_a}\right)^{0.2}\sqrt{\frac{d_a}{in}}\lambda_a\sqrt{\frac{f_c^{'}}{psi}}\left(\frac{c_{a1,s1}}{in}\right)^{1.5} lbf$$

$$V_{b1} = 7 \times \left(\frac{\min\left(5\ \mathrm{in},\ 8 \times 0.5\ \mathrm{in}\right)}{0.5\ \mathrm{in}}\right)^{0.2} \times \sqrt{\frac{0.5\ \mathrm{in}}{1\ \mathrm{in}}} \times 1 \times \sqrt{\frac{3\ \mathrm{ksi}}{0.001\ \mathrm{ksi}}} \times \left(\frac{2.5\ \mathrm{in}}{1\ \mathrm{in}}\right)^{1.5} \times 0.001\ \mathrm{kip}$$

$$V_{b1} = 1.6243\ \mathrm{kip}$$

ACI 318-19 Eq. 17.7.2.2.1b

V_{b2} - Basic single anchor breakout strength condition 2

$$V_{b2} = 9\lambda_a\sqrt{\frac{f_c^{'}}{psi}}\left(\frac{c_{a1,s1}}{in}\right)^{1.5} lbf$$

$$V_{b2} = 9 \times 1 \times \sqrt{\frac{3\ \mathrm{ksi}}{0.001\ \mathrm{ksi}}} \times \left(\frac{2.5\ \mathrm{in}}{1\ \mathrm{in}}\right)^{1.5} \times 0.001\ \mathrm{kip} = 1.9486\ \mathrm{kip}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min\left(\mathrm{V_{b1}},\ \mathrm{V_{b2}}\right) = \min\left(1.6243\ \mathrm{kip},\ 1.9486\ \mathrm{kip}\right) = 1.6243\ \mathrm{kip}$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1

$\Psi_{h,V}$ - Breakout thickness factor

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5\mathrm{c_{a1,s1}}}{\mathrm{t_{conc}}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 2.5\ \mathrm{in}}{8\ \mathrm{in}}},\ 1\right) = 1$$

Calculate concrete breakout capacity at the parallel edge

$\phi = 0.65$ - Concrete shear resistance factor

ACI 318-19 Clause 17.7.2.5.1

$\Psi_{c,V} = 1$ - Breakout cracking factor (shear)

ACI 318-19 Clause 17.7.2.1(c)

$\Psi_{ed,V} = 1$ - Breakout edge effect factor for parallel failure edge

ACI 318-19 Clause 17.7.2.1(c)

$\phi V_{cb\parallel}$ - Design concrete breakout strength in shear of a single anchor (parallel edge)

$$\phi V_{cb\parallel} = 2\phi\left(\frac{A_{Vc}}{A_{Vco}}\right)\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$$

$$\phi V_{cb\parallel} = 2 \times 0.65 \times \left(\frac{23.438\ \mathrm{in}^2}{28.125\ \mathrm{in}^2}\right) \times 1 \times 1 \times 1 \times 1.6243\ \mathrm{kip} = 1.7597\ \mathrm{kip}$$

Result:

PASS = 0.95

DCR - Demand over capacity ratio, comparing two conditions:

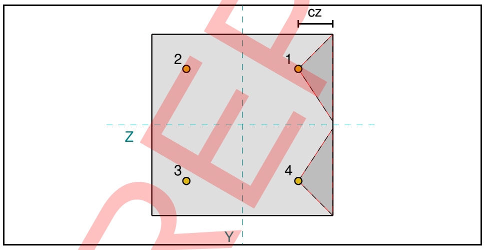
$$DCR = \max\left(\frac{\mathrm{V_{fa\perp}}}{\phi\mathrm{V_{cb\perp}}},\ \frac{\mathrm{V_{fa\parallel}}}{\phi\mathrm{V_{cb\parallel}}}\right)$$

$$DCR = \max\left(\frac{0.75\ \mathrm{kip}}{0.79186\ \mathrm{kip}},\ \frac{0.75\ \mathrm{kip}}{1.7597\ \mathrm{kip}}\right) = 0.94714$$

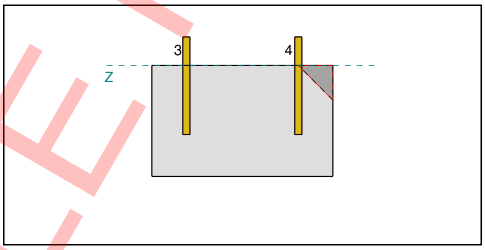
[Go back to summary results table.]

Check No. 6: Concrete Breakout Capacity (Vz Shear)

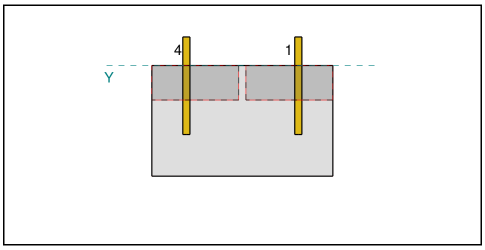
For breakout capacity on failure edge perpendicular to load:



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate shear load per single anchor

$V_z = 1\ \mathrm{kip}$ - Vz shear load

$n_a = 4$ - Total number of anchor rods

$V_{fa\perp}$ - Required concrete breakout shear strength of a single anchor with failure edge perpendicular to load

$$V_{fa\perp} = \frac{V_z}{n_a} = \frac{1\ \mathrm{kip}}{4} = 0.25\ \mathrm{kip}$$

ACI 318-19 Clause 17.7.2.1.2

Determine if the support is a narrow concrete member

The support is not a narrow member.

Symbol	Description	Value
$c_{left,s1}$	Single anchor concrete edge distance (left)	10.500 in
$c_{right,s1}$	Single anchor concrete edge distance (right)	2.500 in
$c_{top,s1}$	Single anchor concrete edge distance (top)	2.500 in
$c_{bottom,s1}$	Single anchor concrete edge distance (bottom)	10.500 in
t_{conc}	Concrete thickness	8.000 in
$c_{a1,s1}$	Single anchor distance to failure edge (+Vz shear)	2.500 in
	Narrow Member	FALSE

Calculation for critical anchor: Anchor ID 1.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.7.2.1.2

$c_{a1,s1} = 2.5 \text{ in}$ - Single anchor distance to failure edge (+Vz shear)

ACI 318-19 Eq. 17.7.2.1.3

A_{Vco} - Maximum projected area for a single anchor

$$A_{Vco} = 4.5(c_{a1,s1})^2 = 4.5 \times (2.5 \text{ in})^2 = 28.125 \text{ in}^2$$

Calculate width of actual projected area on failure surface

ACI 318-19 Clause 17.7.2.1.1

B_{Vc} - Actual length of concrete cone for a single anchor

$$B_{Vc} = \min(\mathrm{c_{top,s1}},\ 1.5\mathrm{c_{a1,s1}}) + \min(\mathrm{c_{bottom,s1}},\ 1.5\mathrm{c_{a1,s1}})$$

$$B_{Vc} = \min(2.5 \text{ in},\ 1.5 \times 2.5 \text{ in}) + \min(10.5 \text{ in},\ 1.5 \times 2.5 \text{ in}) = 6.25 \text{ in}$$

Calculate height of actual projected area on failure surface

ACI 318-19 Clause 17.7.2.1.1

H_{Vc} - Actual height of projected area

$$H_{Vc} = \min(1.5\mathrm{c_{a1,s1}},\ \mathrm{t_{conc}}) = \min(1.5 \times 2.5 \text{ in},\ 8 \text{ in}) = 3.75 \text{ in}$$

Calculate actual projected area

ACI 318-19 Clause 17.7.2.1.1

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc}H_{Vc} = 6.25 \text{ in} \times 3.75 \text{ in} = 23.438 \text{ in}^2$$

Calculate modification factor for lightweight concrete

$\lambda = 1$ - Factor for normal-weight concrete

ACI 318-19 Table 17.2.4.1

λ_a - Modification factor for lightweight concrete

$$\lambda_a = 1.0\lambda = 1 \times 1 = 1$$

Calculate load bearing length of the anchor

$h_{ef} = 5 \text{ in}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 5 \text{ in}$$

Calculate basic single anchor breakout strength

$f_c^{'} = 3 \text{ ksi}$ - Concrete compressive strength (3000)

$d_a = 0.5 \text{ in}$ - Anchor rod diameter

ACI 318-19 Eq. 17.7.2.2.1a

V_{b1} - Basic single anchor breakout strength condition 1

$$V_{b1} = 7\left(\frac{\min(\mathrm{l_e},\ 8\mathrm{d_a})}{d_a}\right)^{0.2}\sqrt{\frac{d_a}{in}}\lambda_a\sqrt{\frac{f_c^{'}}{psi}}\left(\frac{c_{a1,s1}}{in}\right)^{1.5} lbf$$

$$V_{b1} = 7 \times \left(\frac{\min(5 \text{ in},\ 8 \times 0.5 \text{ in})}{0.5 \text{ in}}\right)^{0.2} \times \sqrt{\frac{0.5 \text{ in}}{1 \text{ in}}} \times 1 \times \sqrt{\frac{3 \text{ ksi}}{0.001 \text{ ksi}}} \times \left(\frac{2.5 \text{ in}}{1 \text{ in}}\right)^{1.5} \times 0.001 \text{ kip}$$

$$V_{b1} = 1.6243 \text{ kip}$$

ACI 318-19 Eq. 17.7.2.2.1b

V_{b2} - Basic single anchor breakout strength condition 2

$$V_{b2} = 9\lambda_a\sqrt{\frac{f_c^{'}}{psi}}\left(\frac{c_{a1,s1}}{in}\right)^{1.5} lbf$$

$$V_{b2} = 9 \times 1 \times \sqrt{\frac{3 \text{ ksi}}{0.001 \text{ ksi}}} \times \left(\frac{2.5 \text{ in}}{1 \text{ in}}\right)^{1.5} \times 0.001 \text{ kip} = 1.9486 \text{ kip}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min(\mathrm{V_{b1}},\ \mathrm{V_{b2}}) = \min(1.6243 \text{ kip},\ 1.9486 \text{ kip}) = 1.6243 \text{ kip}$$

Calculate edge effect factor

$c_{a2,s1} = 2.5 \text{ in}$ - Single anchor distance to parallel edge (+Vz shear)

ACI 318-19 Clause 17.7.2.4.1a and 17.7.2.4.1b

$\Psi_{ed,V}$ - Breakout edge effect factor

$$\Psi_{ed,V} = \min\left(1.0,\ 0.7 + 0.3\left(\frac{\mathrm{c_{a2,s1}}}{1.5\mathrm{c_{a1,s1}}}\right)\right) = \min\left(1,\ 0.7 + 0.3 \times \left(\frac{2.5 \text{ in}}{1.5 \times 2.5 \text{ in}}\right)\right) = 0.9$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1

$\Psi_{h,V}$ - Breakout thickness factor

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5\mathrm{c}_{\mathrm{a1,s1}}}{\mathrm{t}_{\mathrm{conc}}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 2.5\ \mathrm{in}}{8\ \mathrm{in}}},\ 1\right) = 1$$

Calculate concrete breakout capacity at the perpendicular edge

$\phi = 0.65$ - Concrete shear resistance factor

ACI 318-19 Clause 17.7.2.5.1

$\Psi_{c,V} = 1$ - Breakout cracking factor (shear)

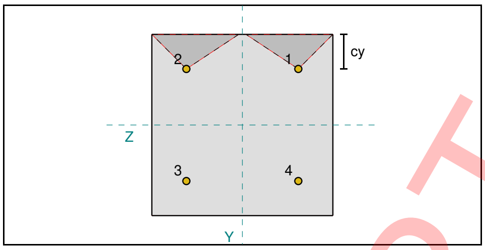
ACI 318-19 Clause 17.7.2.1(a)

$\phi V_{cb\perp}$ - Design concrete breakout strength in shear of a single anchor (perpendicular edge)

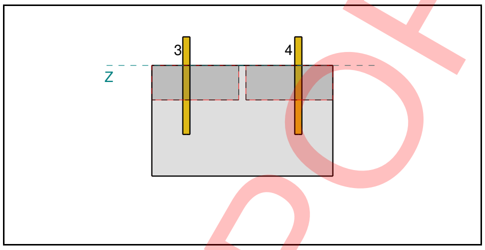
$$\phi V_{cb\perp} = \phi\left(\frac{A_{Vc}}{A_{Vco}}\right)\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$$

$$\phi V_{cb\perp} = 0.65 \times \left(\frac{23.438\ \mathrm{in}^2}{28.125\ \mathrm{in}^2}\right) \times 0.9 \times 1 \times 1 \times 1.6243\ \mathrm{kip} = 0.79186\ \mathrm{kip}$$

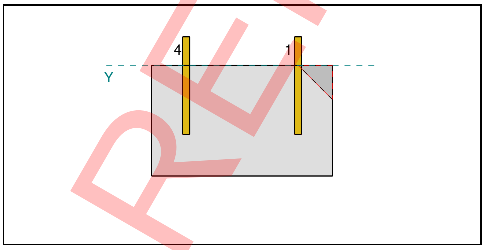
For breakout capacity on failure edge parallel to load:



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate shear load per single anchor

$V_z = 1\ \mathrm{kip}$ - Vz shear load

$n_a = 4$ - Total number of anchor rods

Determine if the support is a narrow concrete member

ACI 318-19 Clause 17.7.2.1.2

The support is not a narrow member.

Symbol	Description	Value
$c_{left,s1}$	Single anchor concrete edge distance (left)	10.500 in
$c_{right,s1}$	Single anchor concrete edge distance (right)	2.500 in
$c_{top,s1}$	Single anchor concrete edge distance (top)	2.500 in
$c_{bottom,s1}$	Single anchor concrete edge distance (bottom)	10.500 in
t_{conc}	Concrete thickness	8.000 in
$c_{a1,s1}$	Single anchor distance to failure edge (+Vy shear)	2.500 in
	Narrow Member	FALSE

Calculation for critical anchor: Anchor ID 1.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.7.2.1.2

$c_{a1,s1} = 2.5\ \mathrm{in}$ - Single anchor distance to failure edge (+Vy shear)

ACI 318-19 Eq. 17.7.2.1.3

A_{Vco} - Maximum projected area for a single anchor

$$A_{Vco} = 4.5(c_{a1,s1})^2 = 4.5 \times (2.5\ \mathrm{in})^2 = 28.125\ \mathrm{in}^2$$

Calculate width of actual projected area on failure surface

ACI 318-19 Clause 17.7.2.1.1

B_{Vc} - Actual length of concrete cone for a single anchor

$$B_{Vc} = \min\left(c_{\text{left,s1}},\ 1.5c_{\text{a1,s1}}\right) + \min\left(c_{\text{right,s1}},\ 1.5c_{\text{a1,s1}}\right)$$

$$B_{Vc} = \min\left(10.5\ \text{in},\ 1.5 \times 2.5\ \text{in}\right) + \min\left(2.5\ \text{in},\ 1.5 \times 2.5\ \text{in}\right) = 6.25\ \text{in}$$

Calculate height of actual projected area on failure surface

ACI 318-19 Clause 17.7.2.1.1

H_{Vc} - Actual height of projected area

$$H_{Vc} = \min\left(1.5c_{\text{a1,s1}},\ t_{\text{conc}}\right) = \min\left(1.5 \times 2.5\ \text{in},\ 8\ \text{in}\right) = 3.75\ \text{in}$$

Calculate actual projected area

ACI 318-19 Clause 17.7.2.1.1

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc} H_{Vc} = 6.25\ \text{in} \times 3.75\ \text{in} = 23.438\ \text{in}^2$$

Calculate modification factor for lightweight concrete

$\lambda = 1$ - Factor for normal-weight concrete

ACI 318-19 Table 17.2.4.1

λ_a - Modification factor for lightweight concrete

$$\lambda_a = 1.0\lambda = 1 \times 1 = 1$$

Calculate load bearing length of the anchor

$h_{ef} = 5\ \text{in}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 5\ \text{in}$$

Calculate basic single anchor breakout strength

$f_c^{'} = 3\ \text{ksi}$ - Concrete compressive strength (3000)

$d_a = 0.5\ \text{in}$ - Anchor rod diameter

ACI 318-19 Eq. 17.7.2.2.1a

V_{b1} - Basic single anchor breakout strength condition 1

$$V_{b1} = 7\left(\frac{\min\left(l_e,\ 8d_a\right)}{d_a}\right)^{0.2} \sqrt{\frac{d_a}{in}} \lambda_a \sqrt{\frac{f_c^{'}}{psi}} \left(\frac{c_{a1,s1}}{in}\right)^{1.5} lbf$$

$$V_{b1} = 7 \times \left(\frac{\min\left(5\ \text{in},\ 8 \times 0.5\ \text{in}\right)}{0.5\ \text{in}}\right)^{0.2} \times \sqrt{\frac{0.5\ \text{in}}{1\ \text{in}}} \times 1 \times \sqrt{\frac{3\ \text{ksi}}{0.001\ \text{ksi}}} \times \left(\frac{2.5\ \text{in}}{1\ \text{in}}\right)^{1.5} \times 0.001\ \text{kip}$$

$$V_{b1} = 1.6243\ \text{kip}$$

ACI 318-19 Eq. 17.7.2.2.1b

V_{b2} - Basic single anchor breakout strength condition 2

$$V_{b2} = 9\lambda_a \sqrt{\frac{f_c^{'}}{psi}} \left(\frac{c_{a1,s1}}{in}\right)^{1.5} lbf$$

$$V_{b2} = 9 \times 1 \times \sqrt{\frac{3\ \text{ksi}}{0.001\ \text{ksi}}} \times \left(\frac{2.5\ \text{in}}{1\ \text{in}}\right)^{1.5} \times 0.001\ \text{kip} = 1.9486\ \text{kip}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min\left(V_{b1},\ V_{b2}\right) = \min\left(1.6243\ \text{kip},\ 1.9486\ \text{kip}\right) = 1.6243\ \text{kip}$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1

$\Psi_{h,V}$ - Breakout thickness factor

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5c_{\text{a1,s1}}}{t_{\text{conc}}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 2.5\ \text{in}}{8\ \text{in}}},\ 1\right) = 1$$

Calculate concrete breakout capacity at the parallel edge

$\phi = 0.65$ - Concrete shear resistance factor

ACI 318-19 Clause 17.7.2.5.1

$\Psi_{c,V} = 1$ - Breakout cracking factor (shear)

ACI 318-19 Clause 17.7.2.1(c)

$\Psi_{ed,V} = 1$ - Breakout edge effect factor for parallel failure edge

ACI 318-19 Clause 17.7.2.1(c)

$\phi V_{cb\parallel}$ - Design concrete breakout strength in shear of a single anchor (parallel edge)

$$\phi V_{cb\parallel} = 2\phi\left(\frac{A_{Vc}}{A_{Vco}}\right) \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$$

$$\phi V_{cb\parallel} = 2 \times 0.65 \times \left(\frac{23.438\ \text{in}^2}{28.125\ \text{in}^2}\right) \times 1 \times 1 \times 1 \times 1.6243\ \text{kip} = 1.7597\ \text{kip}$$

Result:

PASS = 0.32

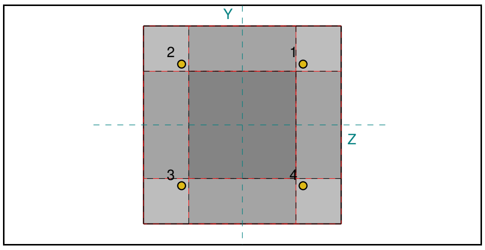
DCR - Demand over capacity ratio, comparing two conditions:

$$DCR = \max\left(\frac{V_{\text{fa}\perp}}{\phi V_{\text{cb}\perp}},\ \frac{V_{\text{fa}\parallel}}{\phi V_{\text{cb}\parallel}}\right)$$

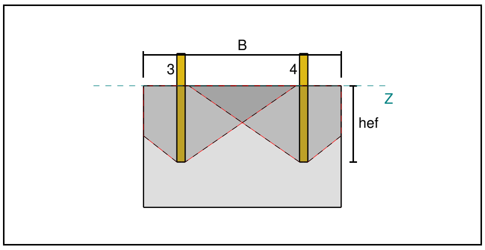
$$DCR = \max\left(\frac{0.25\ \text{kip}}{0.79186\ \text{kip}},\ \frac{0.25\ \text{kip}}{1.7597\ \text{kip}}\right) = 0.31571$$

Go back to summary results table.

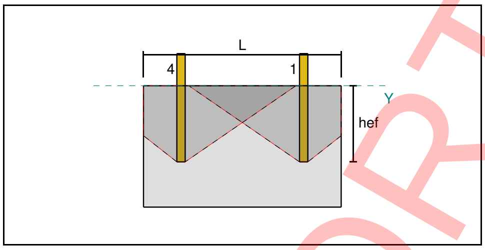
Check No. 7: Concrete Pryout Capacity



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total shear load on anchor group

$V_y = 3\ \text{kip}$ - Vy shear load

$V_z = 1\ \text{kip}$ - Vz shear load

V_{ua} - Resultant shear load

$$V_{ua} = \sqrt{\left((V_y)^2\right) + \left((V_z)^2\right)} = \sqrt{\left((3\ \text{kip})^2\right) + \left((1\ \text{kip})^2\right)} = 3.1623\ \text{kip}$$

$n_a = 4$ - Total number of anchor rods

$n_{a,g1} = 4$ - Number of anchors in anchor group PO 1

V_{ua} - Required concrete pryout strength of an anchor group

$$V_{ua} = \left(\frac{V_{ua}}{n_a}\right) n_{a,g1} = \left(\frac{3.1623\ \text{kip}}{4}\right) \times 4 = 3.1623\ \text{kip}$$

Determine if the support is a narrow concrete member

ACI 318-19 Clause 17.7.2.1.2

The support is classified as a narrow member. The modified values below will be used throughout the calculations.

Symbol	Description	Value
$c_{left,g1}$	Anchor group PO 1 concrete edge distance (left)	2.500 in
$c_{right,g1}$	Anchor group PO 1 concrete edge distance (right)	2.500 in
$c_{top,g1}$	Anchor group PO 1 concrete edge distance (top)	2.500 in
$c_{bottom,g1}$	Anchor group PO 1 concrete edge distance (bottom)	2.500 in
h_{ef}	Anchor rod effective embedment length	5.000 in
$s_{sum,y,g1}$	Anchor group PO 1 sum of spacing along Y-axis	8.000 in
$s_{sum,z,g1}$	Anchor group PO 1 sum of spacing along Z-axis	8.000 in
$n_{y,g1}$	Number of anchors along Y-axis for anchor group PO 1	2.000
$n_{z,g1}$	Number of anchors along Z-axis for anchor group PO 1	2.000
	Narrow Member	TRUE
$h'_{ef,g1}$	Anchor group PO 1 modified effective embedment length	2.667 in

Calculation for critical anchor group: Anchor Group PO 1.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.6.2

A_{Nco} - Maximum projected area for a single anchor

$$A_{Nco} = 9\left(h'_{ef,g1}\right)^2 = 9 \times (2.6667\ \text{in})^2 = 64\ \text{in}^2$$

Calculate length of actual projected cone area (along Z-direction)

$s_{sum,z,g1} = 8\ \text{in}$ - Anchor group PO 1 sum of spacing along Z-axis

$n_{z,g1} = 2$ - Number of anchors along Z-axis for anchor group PO 1

ACI 318-19 Clause 17.6.2

L_{Nc} - Actual length of concrete cone for an anchor group

$$L_{Nc} = \min\left(c_{left,g1},\ 1.5h'_{ef,g1}\right) + \left(\min\left(s_{sum,z,g1},\ 3h'_{ef,g1}\left(n_{z,g1} - 1\right)\right)\right) + \min\left(c_{right,g1},\ 1.5h'_{ef,g1}\right)$$

$$L_{Nc} = \min(2.5\text{ in},\ 1.5 \times 2.6667\text{ in}) + (\min(8\text{ in},\ 3 \times 2.6667\text{ in} \times (2 - 1))) + \min(2.5\text{ in},\ 1.5 \times 2.6667\text{ in})$$

$$L_{Nc} = 13\text{ in}$$

Calculate width of actual projected cone area (along Y-direction)

$s_{sum,y,g1} = 8$ in - Anchor group PO 1 sum of spacing along Y-axis

$n_{y,g1} = 2$ - Number of anchors along Y-axis for anchor group PO 1

ACI 318-19 Clause 17.6.2 B_{Nc} - Actual width of concrete cone for an anchor group

$$B_{Nc} = \min\left(c_{top,g1},\ 1.5h'_{ef,g1}\right) + \left(\min\left(s_{sum,y,g1},\ 3h'_{ef,g1}\left(n_{y,g1} - 1\right)\right)\right) + \min\left(c_{bottom,g1},\ 1.5h'_{ef,g1}\right)$$

$$B_{Nc} = \min(2.5\text{ in},\ 1.5 \times 2.6667\text{ in}) + (\min(8\text{ in},\ 3 \times 2.6667\text{ in} \times (2 - 1))) + \min(2.5\text{ in},\ 1.5 \times 2.6667\text{ in})$$

$$B_{Nc} = 13\text{ in}$$

Calculate actual projected area of the anchor group

$n_{a,g1} = 4$ - Number of anchors in anchor group PO 1

ACI 318-19 Clause 17.6.2 A_{Nc} - Actual projected area for an anchor group

$$A_{Nc} = \min\left(n_{a,g1}A_{Nco},\ L_{Nc}B_{Nc}\right) = \min\left(4 \times 64\text{ in}^2,\ 13\text{ in} \times 13\text{ in}\right) = 169\text{ in}^2$$

Calculate modification factor for lightweight concrete

$\lambda = 1$ - Factor for normal-weight concrete

ACI 318-19 Table 17.2.4.1 λ_a - Modification factor for lightweight concrete

$$\lambda_a = 1.0\lambda = 1 \times 1 = 1$$

Calculate basic single anchor breakout strength

$f_c' = 3$ ksi - Concrete compressive strength (3000)

ACI 318-19 Clause 17.6.2.2.1 $k_c = 24$ - Factor for cast-in anchors breakout strength

ACI 318-19 Eq. 17.6.2.2.1 N_b - Basic single anchor breakout strength

$$N_b = k_c \lambda_a \sqrt{\frac{f_c'}{psi}} \left(\frac{h'_{ef,g1}}{in}\right)^{1.5} lbf$$

$$N_b = 24 \times 1 \times \sqrt{\frac{3\text{ ksi}}{0.001\text{ ksi}}} \times \left(\frac{2.6667\text{ in}}{1\text{ in}}\right)^{1.5} \times 0.001\text{ kip} = 5.7243\text{ kip}$$

Calculate minimum edge distance of anchor

$c_{a,min}$ - Shortest edge distance

$$c_{a,min} = \min\left(c_{left,g1},\ c_{right,g1},\ c_{top,g1},\ c_{bottom,g1}\right) = \min(2.5\text{ in},\ 2.5\text{ in},\ 2.5\text{ in},\ 2.5\text{ in}) = 2.5\text{ in}$$

Calculate eccentricity factor

$e'_N = 0$ - Eccentricity of the resultant tensile force (assumed 0 for shear)

ACI 318-19 Clause 17.6.2.4.1 $\Psi_{ec,N}$ - Breakout eccentricity factor

$$\Psi_{ec,N} = \min\left(1.0,\ \frac{1}{1 + \frac{2e'_N}{3h'_{ef,g1}}}\right) = \min\left(1,\ \frac{1}{1 + \frac{2 \times 0}{3 \times 2.6667\text{ in}}}\right) = 1$$

Calculate edge effect factor

ACI 318-19 Clause 17.6.2.4.1 $\Psi_{ed,N}$ - Breakout edge effect factor

$$\Psi_{ed,N} = \min\left(1.0,\ 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h'_{ef,g1}}\right)\right) = \min\left(1,\ 0.7 + 0.3 \times \left(\frac{2.5\text{ in}}{1.5 \times 2.6667\text{ in}}\right)\right) = 0.8875$$

Calculate nominal concrete breakout capacity

ACI 318-19 Clause 17.6.2.5.1 $\Psi_{c,N} = 1$ - Breakout cracking factor (tension)

ACI 318-19 Clause 17.6.2.6.2 $\Psi_{cp,N} = 1$ - Breakout concrete splitting factor

ACI 318-19 Eq. 17.6.2.1b N_{cbg} - Nominal concrete breakout strength in shear of an anchor group

$$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nco}}\right)\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$$

$$N_{cbg} = \left(\frac{169\text{ in}^2}{64\text{ in}^2}\right) \times 1 \times 0.8875 \times 1 \times 1 \times 5.7243\text{ kip} = 13.415\text{ kip}$$

Calculate design concrete pryout capacity

$\phi = 0.65$ - Concrete shear resistance factor

ACI 318-19 Clause 17.6.2.1 $k_{cp} = 2$ - Factor for pryout strength

ACI 318-19 Eq. 17.7.3.1b ϕV_{cpg} - Design concrete pryout strength of an anchor group

$$\phi V_{cpg} = \phi k_{cp} N_{cbg} = 0.65 \times 2 \times 13.415\text{ kip} = 17.44\text{ kip}$$

Result: **PASS = 0.18**

	DCR - Demand-to-Capacity Ratio $$DCR = \frac{V_{ua}}{\phi V_{cpg}} = \frac{3.1623\ \text{kip}}{17.44\ \text{kip}} = 0.18132$$	Go back to summary results table.

Check No. 8: Anchor Rod Shear Capacity

Calculate portion of Vy shear load on most critical anchor

$V_y = 3\ \text{kip}$ - Vy shear load

$n_a = 4$ - Total number of anchor rods

$V_{ua,y}$ - Required shear strength on anchor due to Vy load

$$V_{ua,y} = \frac{V_y}{n_a} = \frac{3\ \text{kip}}{4} = 0.75\ \text{kip}$$

Calculate portion of Vz shear load on most critical anchor

$V_z = 1\ \text{kip}$ - Vz shear load

$n_a = 4$ - Total number of anchor rods

$V_{ua,z}$ - Required shear strength on anchor due to Vz load

$$V_{ua,z} = \frac{V_z}{n_a} = \frac{1\ \text{kip}}{4} = 0.25\ \text{kip}$$

Calculate resultant load on most critical anchor

V_{ua} - Required shear strength on anchor

$$V_{ua} = \sqrt{\left((V_{ua,y})^2\right) + \left((V_{ua,z})^2\right)} = \sqrt{\left((0.75\ \text{kip})^2\right) + \left((0.25\ \text{kip})^2\right)} = 0.79057\ \text{kip}$$

Calculations according to ACI provisions:

Calculate specified tensile strength of anchor

$F_{u\_anc} = 120\ \text{ksi}$ - Anchor rod tensile stress (A325)

$F_{y\_anc} = 92\ \text{ksi}$ - Anchor rod yield stress (A325)

AISC 360-22 Table J3.2, ACI 318-19 Clause 17.6.1.2

f_{uta} - Specified tensile strength of anchor steel

$$f_{uta} = \min\left(0.75\text{F}_{\text{u\_anc}},\ 1.9\text{F}_{\text{y\_anc}},\ 125\right) = \min\left(0.75 \times 120\ \text{ksi},\ 1.9 \times 92\ \text{ksi},\ 125.00\ \text{ksi}\right) = 90\ \text{ksi}$$

Calculate effective cross-sectional area of anchor

$d_a = 0.5\ \text{in}$ - Anchor rod diameter

ASME B1.1-2019 Table 1

$n_t = 13\ \text{in}^{-1}$ - Number of threads per inch

For smaller diameters, anchor rod is assumed to be UNC. For larger diameters (greater than 1"), anchor rod is assumed to be 8UN thread series.

ACI 318-19 Clause R17.6.1.2

$A_{se,V}$ - Effective cross-sectional area of anchor in tension

$$A_{se,V} = \frac{\pi}{4}\left(d_a - \frac{0.9743}{n_t}\right)^2 = \frac{\pi}{4} \times \left(0.5\ \text{in} - \frac{0.9743}{13\ \text{in}^{-1}}\right)^2 = 0.1419\ \text{in}^2$$

Calculate design shear strength of the anchor rod per ACI provisions

$\phi = 0.65$ - Anchor steel shear resistance factor (ACI)

ACI 318-19 Eq. 17.7.1.2b and Clause 17.7.1.2.1

$\phi V_{sa,aci}$ - Design anchor rod steel shear strength per ACI (reduced capacity)

$$\phi V_{sa,aci} = 0.8\phi 0.6 A_{se,V} f_{uta} = 0.8 \times 0.65 \times 0.6 \times 0.1419\ \text{in}^2 \times 90\ \text{ksi} = 3.9845\ \text{kip}$$

Calculations according to AISC provisions:

Calculate nominal shear stress of the anchor rod

AISC 360-22 Table J3.2

F_{nv} - Nominal shear stress of the anchor rod for threads not excluded on shear plane (N)

$$F_{nv} = 0.45F_{u\_anc} = 0.45 \times 120\ \text{ksi} = 54\ \text{ksi}$$

Calculate design shear strength of the anchor rod per AISC provisions

$A_{rod} = 0.19635\ \text{in}^2$ - Anchor rod area

$\phi = 0.75$ - Anchor steel resistance factor (AISC)

AISC 360-22 Eq. J3-1

$\phi R_{nv,aisc}$ - Design anchor rod steel shear strength per AISC

$$\phi R_{nv,aisc} = \phi F_{nv} A_{rod} = 0.75 \times 54\ \text{ksi} \times 0.19635\ \text{in}^2 = 7.9522\ \text{kip}$$

Calculate governing capacity

ACI 318-19 Clause 17.7.1.2 and AISC 360-22 Section J3.7&8

ϕV_n - Governing anchor rod steel shear strength

$$\phi V_n = \min\left(\phi \text{V}_{\text{sa,aci}},\ \phi \text{R}_{\text{nv,aisc}}\right) = \min\left(3.9845\ \text{kip},\ 7.9522\ \text{kip}\right) = 3.9845\ \text{kip}$$

Result:

PASS = 0.20

DCR - Demand-to-Capacity Ratio

$$DCR = \frac{V_{ua}}{\phi V_n} = \frac{0.79057\ \text{kip}}{3.9845\ \text{kip}} = 0.19841$$

Go back to summary results table.